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**RELATIVE PERMEABILITY MEASUREMENTS
FOR TWO PHASE FLOW THROUGH
LOW PERMEABILITY SOILS**

by

Michael Cunningham

**A thesis submitted to the
Faculty of Graduate Studies and Research
in partial fulfilment of the requirements
for the degree of
Master of Engineering***

**Department of Civil and Environmental Engineering
Carleton University
Ottawa, Ontario, Canada
October, 1994**

*** The Master of Engineering Program
is a joint program with University of Ottawa,
administered by the Ottawa-Carleton Institute for Civil Engineering**

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FOR TWO PHASE FLOW THROUGH
LOW PERMEABILITY SOILS**

submitted by Michael Cunningham
in partial fulfilment of the requirements for
the Degree of Master of Engineering

Professor K.T. Law, Thesis Supervisor

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Department of Civil and Environmental Engineering

Carleton University

October, 1994

ABSTRACT

An experimental investigation was undertaken to measure the relative permeability for two phase flow of water and an organic liquid through a clayey soil. Permeation of clay soil with low dielectric constant organic chemicals is known to result in structural changes and permeability increases. The flow of immiscible liquids through soil is described by multiphase flow principles, one of which is relative permeability. A relative permeability permeameter was designed, taking into account boundary effects which occur in relative permeability testing. Initial results of oil relative permeabilities exceeding unity seemed to indicate that chemical interaction was occurring. However, nonuniform fluid saturations across the cross section were likely causing erroneous results. The results of a lower gradient test indicate that chemical interaction probably did not occur.

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CHAPTER 1

INTRODUCTION

The disposal of organic wastes and the migration of organic chemicals through soil are two important areas of current research. The design of waste disposal facilities is generally the concern of geotechnical/geoenvironmental engineers. The migration of organic contaminants through natural soils is the concern of hydrogeologists. Although the problems are very similar, engineers and hydrogeologists have worked from two very different perspectives.

Many geotechnical engineers have studied the effects of organic chemicals on clay soil permeability. The permeation of an organic chemical through a clay soil can increase the permeability due to changes in the soil structure. The structure of a clay soil is sensitive to the dielectric constant of the pore fluid. Organic chemicals have lower dielectric constants than water. Laboratory permeability tests have been used to study these permeability increases. However, the analysis of these tests has generally been from the perspective of Darcy's law for one phase flow.

A current topic of research among hydrogeologists is the flow of immiscible fluids through soil. Many cases of groundwater contamination are the result of the release of water insoluble organic chemicals to the subsurface environment. A major topic in contaminant hydrogeology is the modelling of the migration of these organic chemicals

using multiphase flow principles. The theory describing multiphase flow was originally developed by petroleum engineers interested in the production of petroleum from natural reservoirs.

The objective of this research is to consider the migration of organic chemicals from both perspectives. The modelling of the migration of organic chemicals through clay landfill liners or natural soils with some clay content must consider both:

- Clay soil-organic chemical interaction and its effects on permeability
- Multiphase flow concepts.

The scope of this research is limited to attempting to measure the relative permeability for the flow of an organic chemical through a clayey soil. Relative permeability is one of the parameters used to describe multiphase flow.

The following three chapters provide a summary of the basic concepts involved in:

- Clay-organic chemical interaction
- Multiphase flow
- Relative permeability testing.

These chapters provide the multidisciplinary background necessary to understand the analytical modelling and experimental testing portions of this research. Analytical modelling was undertaken to aid in the design of an apparatus with which to measure the relative permeability for two phase flow through a fine grained soil. The analytical modelling, the design of the apparatus, and the results of the laboratory testing are

presented.

CHAPTER 2

EFFECT OF ORGANIC CHEMICALS ON THE PERMEABILITY OF CLAY SOILS

2.1 INTRODUCTION

It is estimated that 90% of all industrial hazardous wastes disposed of in the United States are produced as liquids. Approximately 60% of these wastes are organic (Brown and Thomas 1984). The disposal of these wastes has required the extensive engineering of their containment facilities (Evans *et al.* 1983). The hydraulic conductivity of the compacted soil liners used in many of these facilities is crucial to the control of contaminant releases to the environment (Acar *et al.* 1985). Most state environmental regulations require a hydraulic conductivity no greater than 1×10^{-7} cm/s. These permeabilities are generally determined using relatively pure water (Anderson 1982). As early as 1942, researchers found that the hydraulic conductivity of clay soil to benzene was up to one million times greater than that to water (Mesri and Olson 1971). Obviously, accurate prediction of compacted clay liner performance requires an understanding of the interaction between organic chemicals and clay liners (Evans *et al.* 1983). Cases of industrial landfills where waste/soil interaction may have led to increased contaminant release have been reported. Herzog *et al.* (1989) reported the release of organic chemicals from an industrial waste landfill in Illinois. Organic pollutants had migrated 15 m in only three years. This amounted to a leakage rate 100 to 1000 times greater than that predicted using the hydraulic conductivity of the natural clay till to

water. This is a case where interaction between organic chemicals and the natural soil (not a liner) may have occurred. This particular landfill relied on the natural attenuation of the surrounding clay till to contain the hazardous waste. In the design of a landfill liner, or in trying to predict contaminant migration through a natural clayey soil, it is important to recognize the interaction between organic chemicals and clay soil permeability.

The purpose of this chapter is to summarize some of the major findings about the effect of organic chemicals on the permeability of clay soils. In order to discuss the effect of industrial chemicals on permeability, some background information is provided on:

- the various types of industrial chemicals
- a review of clay mineralogy
- methods of testing for waste/liner compatibility.

2.2 TYPES OF INDUSTRIAL CHEMICALS

The United States Environmental Protection Agency (USEPA) classifies industrial chemicals into four physical classes (Evans *et al.* 1983):

- organic
- aqueous organic
- aqueous inorganic
- sludges.

Organic industrial wastes are those in which the solvent is an organic liquid, and may

contain other organic chemicals (solutes) dissolved in the organic solvent (Evans *et al.* 1983). These wastes are very common. A list of typical chemicals found in the drums of industrial landfills is given in Table 2.1. Typical sources of these organic wastes are pesticide manufacturing wastes, spent cleaning solvents, and petroleum distillates (Evans *et al.* 1983).

Aqueous organic industrial wastes are liquids where water is the solvent and the solutes are predominantly organic. Examples of these wastes are those from wood preserving operations, dye wastes, and wastes from the production of pesticides or ethylene glycol (Evans *et al.* 1983). In some literature, these liquids are referred to as dilute organic chemicals. Any chemical may be present up to its solubility limit (Anderson and Jones 1983).

Aqueous inorganic industrial wastes are liquids where water is the solvent and the solute is mostly inorganic. Examples of these wastes are brines, caustic sodas, inorganic acids, and inorganic salts (Evans *et al.* 1983). These chemicals can also have serious deleterious effects on the hydraulic conductivity of clay soils, but they will not be discussed here.

The fourth type of industrial chemical is sludges. There is very little, if any, published material on the effects of sludges on landfill liners.

It is the first two types of industrial chemicals (organic and aqueous organic) that are of

Table 2.1 Typical contents of drums in industrial landfills (after Anderson 1982).

Dibromo propanol	Urethane lacquer
Methylene chloride bottoms	Toluene still bottoms
Fractional bottoms	Methanol slurry
Pesticides	Resinous sludge
Distillation bottoms	Phenol tar sludge
PCB waste	Fuel oil sludge
Oil sludge	Contaminated fuel oil
Chlorinated still bottoms	Glycol waste
Trichlor bottoms	Heat transfer oil sludge
Ink sludge	Acetic acid sludge
Phenol sludge	MEK distillation bottoms
Paint sludge	Liquid resins
Tank sludge	Freon bottoms
Still residues	Oil and grease sludge
Organic residues	Miscellaneous liquid wastes
Terephthalate sludge	Chlorinated xylene sludge
PCB bottoms	Benzyl alcohol bottoms
Laboratory chemicals	Grinding oil sludge
Sulfonated still bottoms	Filter sludge
Heavy metal sludge	Transformer oil sludge
Chlorinated solvent sludge	Methyl chloroform bottoms
Mercury filter press sludge	Organic sludges
Tank bottom residues	Perchlor bottoms
Still bottom residues	Waste solvents
Chlorinated organic residues	Resins and solvents
Plating sludge	Mixed solvents

interest in this discussion. The organic industrial wastes can be further subdivided into (Evans *et al.* 1983):

- organic acids
- organic bases
- neutral nonpolar fluids
- neutral polar fluids.

Organic acids are proton donors (such as acetic acid), while organic bases are proton acceptors (such as aniline) (Evans *et al.* 1983). While the acid/base nature of these chemicals can have additional implications for clay soil permeability, it is the next two types of organic chemicals that are of interest here.

Neutral nonpolar organic fluids have no charge and little, if any, dipole moment (Evans *et al.* 1983). Most *nonaqueous* liquids disposed of in industrial landfills are of this type. In other words, these chemicals are typically present as a separate liquid phase. These chemicals do, however, have very limited solubilities and can be found as dilute solute plumes. Examples of neutral nonpolar organic fluids are oils and many discarded industrial solvents. These fluids are commonly aromatic compounds such as benzene, toluene, and xylene, as well as aliphatic compounds such as heptane (Anderson and Jones 1983). These examples are less dense than water. Another common type of waste liquid is chlorinated hydrocarbons, such as trichloroethylene (TCE). Chlorinated hydrocarbons are typically denser than water (Fetter 1992).

Fig. 2.1 Types of industrial chemicals (after Anderson and Jones 1983).

Neutral polar organic fluids have no charge and relatively strong dipole moments (Evans et al. 1983). These are common industrial feedstocks and solvents. These chemicals typically have a high water solubility due to their polarity, but are often disposed of in concentrated form (Anderson and Jones 1983). Due to their high water solubility, these chemicals are found as solutes when they contaminate groundwater.

A schematic classification scheme for liquid industrial wastes is shown in Fig. 2.1. To understand the effects of organic and aqueous organic chemicals on clay soil permeability, an understanding of clay mineralogy is necessary.

2.3 CLAY MINERALOGY

Clay soils are composed of crystalline particles. Each particle is typically less than 2 μm in diameter. The crystals are plate-like in shape and generally composed of four elements (Das 1990):

- oxygen
- silicon
- aluminum
- hydrogen.

The silicon and oxygen form a silica tetrahedron which is repeated to form a planar sheet called the silica or tetrahedral sheet. The aluminum, hydrogen, and oxygen form an octahedron. The aluminum is commonly replaced by iron or magnesium at some sites. The octahedron is repeated to form a planar sheet called the octahedral or gibbsite sheet

Fig. 2.2 a) Silica tetrahedron; b) silica (tetrahedral) sheet; c) alumina octahedron; d) octahedral (gibbsite) sheet; e) stacked silica and octahedral sheets (after Grim 1959).

(Das 1990). These two types of sheets get stacked together as shown in Fig. 2.2. Positive ions are often replaced by other cations of lower valence, giving the sheets a net negative charge. This replacement process is called isomorphous substitution (Oweis and Khera 1990).

The different clay minerals are formed by different stacking arrangements. The most common clay minerals are kaolinite, illite and montmorillonite which are shown in Fig. 2.3. Both illite and montmorillonite have an "interlayer" space which is filled by potassium ions and water, respectively. Expansion of this interlayer space upon wetting makes montmorillonite an expansive clay (Oweis and Khera 1990).

A series of stacked layers make up one clay *particle*. Due to the net negative charge, clay particles have adsorbed cations on their surface. Upon saturation in water, these cations tend to diffuse away but are still attracted to the clay particle. This forms the diffuse double layer shown in Fig. 2.4. This diffuse double layer of cations surrounding the clay particle has an associated electric potential which decreases with distance from the particle. The potential is described according to the Gouy-Chapman theory by the following (Fernandez and Quigley 1985):

$$\psi_x = \psi_o e^{-kx} ; \quad k = \sqrt{(8 \pi e \sum n_i z_i^2) / \epsilon k T} \quad [2.1]$$

Where ψ_x = potential at a distance x from the clay particle

ψ_o = potential at the surface of the clay particle

ϵ = dielectric constant of the pore fluid

Fig. 2.3 Structure of a) Kaolinite; b) Illite; c) Montmorillonite (after Das 1990).

Fig. 2.4 Diffuse double layer (after Das 1990).

k = Boltzman's constant

T = absolute temperature (K)

e = elementary charge (4.803×10^{-10} esu)

n_i = number of ions per unit volume of bulk pore fluid

z_i = valence of cation (i)

x = distance from the clay particle surface.

The potential as a function of distance from the clay particle surface is shown in Fig. 2.5. The distribution is shown for several dielectric constants representative of different pore fluids. More complicated expressions are available to describe the clay-water-electrolyte system (see for example Yong *et al.* 1992). However, the simple Gouy-Chapman diffuse double layer model is the most popular.

The diffuse double layer has significant impact on the interaction between clay particles. Attractive forces between clay particles are mostly van der Waal's forces. The repulsive forces are primarily due to the interaction between the diffuse double layers of adjacent clay particles. The positively charged diffuse double layers repel each other. Thus, any decrease in the diffuse double layer thickness increases the net attractive force between clay particles (Acar *et al.* 1985). Consequently, the interparticle spacing decreases. The equilibrium interparticle spacing is given by (Brown and Daniel 1988):

$$H = \left(\frac{\epsilon k T}{8 \pi h_o e z} \right)^{1/2} \quad [2.2]$$

Where h_o = electrolyte concentration

Fig. 2.5 Electric potential (ψ_x) as a function of distance from the clay particle surface for various pore fluids (after Fernandez and Quigley 1985).

z = cation valence.

The Gouy-Chapman theory applies only to dilute clay suspensions. However, the behaviour of *compacted* clay can be qualitatively described by the same theory (Brown and Daniel 1988).

There are two extremes of clay structure. A *flocculated* structure involves edge to face contacts. A structure with all particles more or less parallel is a *dispersed* structure (Das 1990). Both of these structures are shown in Fig. 2.6. These two structures result from different conditions during initial deposition from suspension. If the diffuse double layer thickness is small while the clay is in suspension, then the attractive forces dominate, the interparticle spacing is small, and a flocculated structure forms on deposition. By the opposite process, a thick diffuse double layer results in a dispersed structure (Anderson and Jones 1983).

A flocculated soil structure has large pores and thus has a larger hydraulic conductivity than a dispersed structure (Acar and Ghosh 1986). A summary of the effects on clay structure due to conditions during deposition is given in Table 2.2.

As stated previously, although this theory is applicable to dilute suspensions, it is valuable as a tool for qualitatively describing the behaviour of compacted clay when permeated with various liquids (Brown and Daniel 1988).

Fig. 2.6 Clay structures. a) dispersed; b) flocculated (after Lambe 1958b).

Table 2.2 Effect of pore fluid parameters on double layer thickness and soil structure (after Evans *et al.* 1985).

PORE FLUID PARAMETER	CHANGE IN PORE FLUID PARAMETER	CHANGE IN DOUBLE LAYER THICKNESS	CHANGE IN SOIL STRUCTURE TENDENCY
DIELECTRIC CONSTANT	INCREASE	INCREASE	DISPERSED
	DECREASE	DECREASE	FLOCCULATED
ELECTROLYTE CONCENTRATION	INCREASE	DECREASE	FLOCCULATED
	DECREASE	INCREASE	DISPERSED
ION VALENCE	INCREASE	DECREASE	FLOCCULATED
	DECREASE	INCREASE	DISPERSED
pH	INCREASE	INCREASE	DISPERSED
	DECREASE	DECREASE	FLOCCULATED

The ability of compacted clay to interact with the pore fluid has serious implications when testing for waste/liner compatibility. To understand these implications, a knowledge of the testing methodology is necessary.

2.4 METHODS OF TESTING FOR CLAY/ORGANIC LIQUID INTERACTION

The function of landfill liners is to provide as impermeable a layer as possible (Fluet *et al.* 1992). Consequently, liner *failure* is defined as an increase in permeability (Peterson and Gee 1985). Tests for waste/liner compatibility measure the hydraulic conductivity of the liner to the waste fluid (Bowders *et al.* 1985). The two main types of apparatuses for permeability testing are:

- rigid wall (compaction mold) permeameters
- flexible wall permeameters.

These are shown in Fig. 2.7.

The rigid wall (compaction mold) permeameter uses the standard ASTM mold for Proctor density determination (ASTM D-698). The disadvantage of this test is that there is no confining stress. Any soil shrinkage on permeation with the waste liquid can lead to side wall leakage and incorrect results due to incorrectly high flow rate measurements (Bowders *et al.* 1985).

The flexible wall permeameter uses a modified triaxial cell apparatus that is usually used for measuring the strength of clay. The disadvantage of this method is that the high

Fig. 2.7 Waste/liner compatibility test apparatuses. a) Rigid wall (compaction mold) permeameter ; b) Flexible wall permeameter (after Oweis and Khera 1990).

applied confining stress can close shrinkage cracks which might form *in situ*. Thus, the clay may seem less permeable (Bowders *et al.* 1985). One advantage of this method is that a back pressure can be applied to facilitate sample saturation. An additional consideration of this test is that many permeants may react with the membranes, leading to membrane failure. Teflon tape is usually wrapped around the specimen to prevent contact between the permeant and the liner (Oweis and Khera 1990).

The compaction mold permeameter is most suitable for situations where the liner will not be subjected to high confining stresses (Bowders *et al.* 1985). An example would be the liner of a liquid waste impoundment facility (Bowders and Daniel 1987). If the liner will be subjected to high confining stresses which can close shrinkage cracks, then the flexible wall permeameter is more appropriate (Bowders *et al.* 1985). An example would be a solid waste disposal facility (Bowders and Daniel 1987).

An alternative form of the rigid wall permeameter is the double ring permeameter suggested by Anderson *et al.* (1985). This permeameter is shown in Fig. 2.8. This permeameter is a modified version of the rigid wall (compaction mold) permeameter. It provides a method of checking for sidewall leakage around the soil specimen. Flow through the core section is separated from flow near the permeameter sidewalls. The flux (flow rate/cross section area) can be calculated for both sections. If the flux through the outer portion is significantly larger than through the core, it is a likely sign of sidewall leakage.

a)

b)

Fig. 2.8 a) Double ring permeameter; b) base plate (after Anderson *et al.* 1985).

All three of the aforementioned tests are generally performed under an applied head, and the flow rate measured. Fernandez and Quigley (1985) used a permeameter based on one described by Olsen (1966). This permeameter is shown in Fig. 2.9. The fluid is pushed into the soil specimen at a constant flow rate using a syringe mounted in a triaxial testing frame. Triaxial testing frames typically have a large range of gear ratios which can be used to vary the flow rate over several orders of magnitude. This test is a constant flow rate test, also known as a flow pump test (Oweis and Khera 1990). These tests have the advantages that flow measurement is not necessary, and the test length is typically short due to the fast attainment of steady state (Oweis and Khera 1990). The major disadvantage of rigid wall (compaction mold) permeameters is that there is no applied stress. Any shrinkage of the clay leads to sidewall leakage. The permeameter in Fig. 2.9 is similar to a rigid wall permeameter, but a compressed spring is used to apply an overburden stress. This is felt to more accurately model the conditions in the ground. This permeameter is quite useful for volatile, noxious or toxic compounds since the system is completely sealed. Like the rigid wall permeameter, no back pressure is applied to saturate the sample. For this permeameter, the outflow is released into a small vial.

There are some consistent characteristics of permeability tests done with all of these permeameters. The results are usually expressed as permeability as a function of pore volumes of flow. One pore volume is equal to the volume of the pores in the sample. The presentation of flow in pore volumes (P.V.) is a method of normalizing the results for samples of different size. Typical test results are shown in Fig. 2.10 for the

a)

b)

Fig. 2.9 a) Permeameter used by Fernandez and Quigley; b) schematic of cell (1985).

Fig. 2.10 Typical results presentation (after Fernandez and Quigley 1985).

permeation of acetone through a clay soil (Acar and Ghosh 1986). Specimens are typically compacted in the laboratory. Many different methods are used, although the Harvard miniature compaction apparatus is popular. The permeability of the soil specimen to water is often determined with a dilute solution of CaSO_4 (0.01 N is popular). Following saturation, the permeant of interest is forced through. Whether the permeant of interest is an aqueous solution (organic or inorganic) or a pure organic (water soluble or insoluble), that permeant is forced through in a piston type flow. Hydraulic gradients are typically quite high to keep the testing time short. Gradients between 1 and 500 are common (Oweis and Khera 1990), but gradients of 4000 have been reported (Fernandez and Quigley 1991).

2.5 EFFECT OF ORGANIC CHEMICALS ON CLAY PERMEABILITY

As mentioned previously, the soil property of interest is the hydraulic conductivity of the clay. Most states in the United States require that clay liners have a hydraulic conductivity not exceeding 1×10^{-7} cm/s (Anderson 1982). However, even without chemical interaction with the clay, different liquids will have different "hydraulic" conductivities. The "hydraulic" conductivity of any liquid is given by (Green *et al.* 1981):

$$k = K \frac{\gamma}{\eta} \quad [2.3]$$

Where k = hydraulic conductivity

K = intrinsic permeability

γ = unit weight of the permeating liquid

η = dynamic viscosity of the permeating liquid.

The intrinsic permeability is independent of the permeating liquid as long as there is no chemical interaction. Although tests for chemical interaction should be based on changes in intrinsic permeability, any chemical interaction will often swamp the effect (Fernandez and Quigley 1985). Consequently, many tests are based on the traditional hydraulic conductivity alone. It is the fundamental basis of this research that both hydraulic conductivity and intrinsic permeability are insufficient parameters for describing the flow of nonaqueous fluids (typically nonpolar organics like benzene, toluene, xylene, and heptane). The flow of nonaqueous fluids must be described using multiphase flow concepts.

It should be noted that there is not always complete agreement on the effects of some chemicals. Different researchers have found increases and decreases in hydraulic conductivity for the same chemicals. Much of the discrepancy is likely due to the test method (Bowders *et al.* 1985). A good example of this effect is found in work done by Bowders and Daniel (1987). Both a kaolinite and an illite-chlorite clay were permeated with several organic chemicals. One of the chemicals used was trichloroethylene (TCE). Organic-clay interaction was studied using both rigid wall (compaction mold) and flexible wall permeameters. With the rigid wall permeameter, permeation with TCE resulted in permeability *increases* of 140 to 500 times in both types of clay. With the flexible wall permeameter, permeation with TCE resulted in permeability *decreases* of 2 to 3 orders of magnitude. The difference is generally attributed to sidewall leakage in the rigid wall

permeameter when the clay shrinks, leaving gaps between the soil and the wall.

However, the two test methods often do agree on the change in permeability. For example, the same researchers (Bowders and Daniel 1987) tested the same two clays with pure methanol. The permeability increased by 44 times in the rigid wall permeameter and by 1.2 times in the flexible wall permeameter. The two tests disagree only in the magnitude of the permeability change. Permeability tests using double-ring permeameters have also shown permeability increases. Anderson *et al.* (1985) permeated bentonite slurries with xylene. The bentonite slurry was to simulate the filter cake which forms on the side of slurry trenches used to hydraulically isolate contaminated sites. The double ring permeameter is able to separate the sidewall leakage component of the flow. Permeability increases of two orders of magnitude were found using only the flow through the core section.

There can really be very little doubt that organic chemicals do change the permeability of clay soils. Research in the ceramics industry 40 years ago showed that clays can be up to one million times more permeable to organic compounds than to water (Anderson 1982). A veritable plethora of research has been published in recent years on this topic. Most of this work has been performed by engineers interested in the effects of organic chemicals on clay landfill liners. Oweis and Khera (1990) summarized some of this research. Their summary is shown in Table 2.3. This table could be expanded many times over if all of the work published on this topic was included. However, the

Table 2.3 The effect of various organic compounds on the permeability of clay soils (after Oweis and Khera 1990).

COMPOUND	SOIL	CONC.	PERMEA METER	K_s/K_w	REF
HEPTANE	KAOLINITE	PURE	FW RW	0.03 250	1 1
	ILLITE- CHLORITE	53mg/L	RW	660	1
			FW RW	1 1	1 1
BENZENE	NATURAL	PURE	RW	1	2
XYLENE	SMECTITE	99%	RW	500	3
TETRA- CHLORO- METHANE	SAND- BENTONITE MIX	PURE	FW	150	4
		720mg/L	FW	1	4
NITRO- BENZENE	GEORGIA KAOLINITE	PURE	FW	0.003	5
		0.10%	FW	0.68	5
CHLORINA- TED HYDRO- CARBONS	SOIL- BENTONITE MIX	PURE	RW	60	6
	CONTAMIN ATED SOIL	PURE	RW	94	6
METHANOL	KAOLINITE	PURE	FW RW	1.7 7.5	1
	LUFTIN CLAY	PURE	FW	3.8	7
			RW	1000	8
	BENTONITE SOIL MIX	PURE	DR	500	9
PHENOL	BRUNSWICK SOIL	4.7g/L	FW	0.75	10
	WOODBURY CLAY	4.7g/L	FW	0.8	10
ACETONE	SMECTITE	99%	RW	900	2

COMPOUND	SOIL	CONC.	PERMEA METER	K_o/K_w	REF
ACETONE	KAOLINITE	PURE	FW	1.33	11
	SMECTITE SAND MIX	PURE	FW	2.7	11

Notes:

K_o = permeability to organic

K_w = permeability of water

RW = rigid wall permeameter

FW = flexible wall permeameter

DW = double ring permeameter

References:

- 1- Bowders and Daniel (1987)
- 2- Anderson (1982)
- 3- Brown *et al.* (1983)
- 4- Evans *et al.* (1985)
- 5- Acar *et al.* (1985)
- 6- Davis *et al.* (1985)
- 7- Foreman and Daniel (1987)
- 8- Brown and Anderson (1983)
- 9- Anderson *et al.* (1985)
- 10- Korfiatis *et ai.* (1986)
- 11- Acar and C'Hollosy (1987)

conclusions would be the same. It is more important to understand why these permeability changes occur.

Three main reasons have been cited for the increase in permeability. All of these reasons are based on changes in the diffuse double layer. As mentioned previously, the diffuse double layer of cations surrounding a clay particle is described by the Gouy-Chapman theory. According to this theory, the thickness of the diffuse double layer is a function of the dielectric constant of the pore fluid. Most organic chemicals have low dielectric constants compared to that of water (80.4). The dielectric constants of several organics are given in Table 2.4. The relationship between permeability and dielectric constant is shown in Fig. 2.11, based on test results by Fernandez and Quigley (1985). The low dielectric constant of these fluids results in a small double layer thickness when they are introduced into the pores of a clay soil.

The first, and probably least important, reason that the permeability increases is the smaller effective particle size (Lentz *et al.* 1985). The reduced double layer thickness means that less of the pore fluid is immobilized by the cations held in the diffuse double layer. The smaller effective particle size (or larger effective void size) results in a larger permeability. However, according to Lambe (1956), the effects of the reduction in the thickness of immobilized fluid are small in comparison to the effects of structural changes.

Table 2.4 Dielectric constants of several organic chemicals (after Fernandez and Quigley 1985).

COMPOUND	DIELECTRIC CONSTANT
Acetone	21.45
Aniline	6.89
Benzene	2.28
Cyclohexane	2.02
Ethanol	25.0
Methanol	32.6
Isopropanol	18.3
o-Xylene	2.47

Fig. 2.11 Permeability as a function of dielectric constant of pore fluid (after Fernandez and Quigley 1985).

It is these structural changes that are the second reason for permeability increases on permeation with organics. The Gouy-Chapman theory predicts that permeation with low dielectric constant organics leads to a decrease in interparticle repulsion forces. The decrease in interparticle repulsion forces leads to a flocculated structure (Evans *et al.* 1983). The flocculated structure has larger pores and is consequently more permeable.

It is important to note that this structural change of the clay is only possible if the clay is initially *dispersed*. However, clay landfill liners are always compacted on the wet side of the optimum moisture content (based on maximum Proctor density). It was shown by Lambe (1958a) that soils compacted wet of optimum have a lower hydraulic conductivity, which is the objective of clay landfill liners. This lower hydraulic conductivity is achieved by an initially *dispersed* structure (Anderson and Jones 1983). Thus, the clay is able to undergo structural changes on permeation by organics.

The same mechanism is believed to be responsible for the destruction of the plasticity of clays (Daniel *et al.* 1988). Bowders and Daniel (1987) permeated several clays with different organic chemicals. The change in Atterberg limits (liquid limit and plastic limit) and change in permeability were examined. They concluded that if a liquid *does not* alter the Atterberg limits (relative to water) then the liquid will not alter the hydraulic conductivity of the soil. If the Atterberg limits *are* altered, only a hydraulic conductivity test can predict if permeability changes will occur. Consequently, Atterberg limit tests are good initial index tests for waste/liner compatibility.

The third, and perhaps most important, reason for permeability increases is shrinkage of clay soils upon permeation with organics. Green *et al.* (1981), Acar *et al.* (1985), Bowders and Daniel (1987), Acar and Ghosh (1986), and Brown and Daniel (1988) all report shrinkage of clays on permeation with organic chemicals. This shrinkage is generally attributed to collapse of the diffuse double layer. In a compacted liner this could lead to tension cracks and complete liner failure (Bowders and Daniel 1987). A good example of this effect is found in some work by Abdul *et al.* (1990). Two types of clay soils (kaolin and bentonite) were permeated with nine organic chemicals. The aqueous solutions of these organics had little effect. However, the effect of the pure organic was quite dramatic. Soon after permeation with the solvent began, vertical cracks appeared in the specimen. This occurred for both types of soil. The cracks slowly increased in length towards the downgradient end of the specimen. As the cracks reached the end, flow increased dramatically. Cracking is a manifestation of major structural changes. Soil particles pull together into aggregations of many particles resulting in large interaggregate pores (Anderson and Jones 1983). There has, however, been research which disputes the belief that changes occur in the soil fabric. Uppot and Stephenson (1989) permeated both a kaolinite clay and a montmorillonite clay with xylene. A scanning electron microscope was used to determine if the clay structure was the same before and after permeation. No change in structure was found. Similarly, Acar *et al.* (1985) permeated kaolinite with nitrobenzene, acetone, phenol, and benzene. Mercury intrusion tests were done before and after permeation. No change was found in the pore size distribution. In conclusion, many researchers have observed cracking of clay soils

upon permeation with organics. However, there has been little research which could quantitatively define the structural changes in the clay.

Some research has shown that certain soils may be more susceptible to chemical interaction with organic chemicals than other soils. Currently, high activity soils are used for the low permeability layer in landfill liners (Acar *et al.* 1985). Activity, A , is defined as (Das 1990):

$$A = \frac{PI}{\text{weight percent of clay-size } (<2\mu\text{m}) \text{ fraction}} \quad [2.4]$$

Where PI = Plasticity index

Montmorillonite is a high activity clay mineral. Kaolinite is a low activity clay mineral. Acar and Ghosh (1986) permeated several clay soils of different activities with acetone. The ratio of permeability to acetone to permeability to water (K_a/K_w) was plotted as a function of activity, as shown in Fig. 2.12. The high activity soils experienced less permeability increase.

Another factor to consider is the difference in the response to dilute organic solutions and the pure organic. Fernandez and Quigley (1988) studied the effect of the concentration in aqueous solution of water soluble organics on the permeability of clay. A typical example is shown in Fig. 2.13 for the permeation of a brown Samia clay with an aqueous ethanol solution. Increasing ethanol concentrations initially *decreased* the hydraulic conductivity. The hydraulic conductivity only exceeded the original value at

Fig. 2.12 Effect of activity on permeability changes (after Acar and Ghosh 1986)

K_a = permeability to acetone

K_w = permeability to water.

Fig. 2.13 Effect of ethanol concentration on hydraulic conductivity and intrinsic permeability (after Fernandez and Quigley 1988).

greater than 70% ethanol (wt/wt). A more informative way of considering the data is to look at intrinsic permeability changes. As shown in Fig. 2.14, the viscosity of the solution actually peaks at approximately 50% ethanol. Using the viscosity data, the intrinsic permeability was calculated and is shown in Fig. 2.13. The intrinsic permeability never decreases. The intrinsic permeability is actually constant up to roughly 50 % ethanol. At this point, it starts to increase. The increase in intrinsic permeability can be attributed to the steadily decreasing dielectric constant with increasing ethanol content, as shown in Fig. 2.15. The limited effect of dilute organic solutions has been confirmed by many other researchers (Daniel *et al.* 1988; Evans *et al.* 1985).

A recurring theme in the discussion of waste/liner compatibility tests is the use of a test which correctly models the *in situ* conditions. The interest is primarily on whether cracks which could form upon permeation with organics will close or not. The rigid wall permeameter test is excessively conservative because cracks are free to form and give unreasonably high permeabilities. The flexible wall permeameter is perhaps dangerously unrealistic because any cracks which would form on shrinkage are immediately closed by the cell pressure. The permeameter used by Fernandez and Quigley (1985) is perhaps the best one to model the *in situ* state. This same permeameter was used to determine the impact of effective stress on controlling the effects of clay-organic liquid interaction (Fernandez and Quigley 1991). The major conclusion of this work was that vertical effective stress (prior to, and during permeation) permitted the soil to consolidate and yield laterally as it shrunk on permeation with organic chemicals. In other words, as the

Fig. 2.14 Viscosity as a function of hydrocarbon content (after Fernandez and Quigley 1988).

Fig. 2.15 Dielectric constant as a function of hydrocarbon content (after Fernandez and Quigley 1988)

double layer collapses and the clay shrinks, the clay consolidates, and any macropores remain closed. No shrinkage cracks form. No increase in hydraulic conductivity occurs.

There is one last important point that must be made. Very little distinction is made in many of these tests as to whether the permeant is a nonaqueous phase liquid, a water soluble pure phase, or a solute. This information is generally provided, but very little recognition is given to the different modes of permeation. Solute travel is described by the advection-dispersion equation. Nonaqueous phase transport is described by multiphase flow concepts discussed in the next chapter. Very few researchers who have published on the topic of clay/permeant interaction have expressed any knowledge of this important difference.

Interestingly, Lambe (1956) did recognize both the clay/organic and multiphase flow components of this problem. This very early recognition was the result of an interesting project executed by the ESSO corporation while working in Venezuela. Additional reservoir capacity was needed to store 3 500 000 barrels of fuel oil. There was insufficient time to construct conventional steel tankage. It was decided to build a 4 000 000 barrel capacity clay lined reservoir in which to store the fuel oil. Lambe considered clay/fuel oil interaction due to the low dielectric constant of the fuel oil. Lambe also considered multiphase flow concepts since fuel oil is immiscible with water. These multiphase flow concepts, originally developed by petroleum engineers, are discussed in the next chapter.

Unfortunately, recent work by engineers interested in the design of clay liners for waste impoundments have paid little consideration to multiphase flow concepts. Both Bowders and Daniel (1987) and Uppot and Stephenson (1989) mention that immiscible fluids must exceed an entrance pressure in order to initiate flow. Fernandez and Quigley (1985) give perhaps the best consideration of multiphase flow. Approximately 2 pore volumes of water insoluble benzene was forced through brown Samia clay. There was little change in permeability. The pore fluid was extracted and found to be only 8 % benzene. The benzene was thought to have passed through interconnected macropores only. An interesting subsequent test involved three stage permeation: water, ethanol, and benzene. The permeation sequence involved 1 pore volume of water flow, followed by 3.0 pore volumes of ethanol, followed by 4.0 pore volumes of benzene. This resulted in very large hydraulic conductivity increases from 7×10^{-8} cm/s initially to 10^{-4} cm/s. The pore fluid at the end of the test was 30 % benzene, 35 % ethanol, and 35 % water. The water soluble ethanol is initially able to invade many water filled pores. The ethanol soluble benzene is then able to invade many ethanol saturated pores. The migration of benzene in this test is more that of a solute than that of a separate phase. The increased benzene saturation results in greater permeability increases.

2.6 CONCLUSIONS

There has been much research over the past decade showing the reaction between clay soils and organic chemicals. This research has shown that permeation with low dielectric constant organic chemicals results in decreased double layer thickness. The decreased

double layer thickness results in increased permeability due to:

- the increased pore size
- flocculation of the clay
- cracking and macropore development.

CHAPTER 3

TRANSPORT OF NONAQUEOUS PHASE LIQUIDS - MULTIPHASE FLOW

3.1 INTRODUCTION

There are two main types of groundwater contaminants (Domenico and Schwartz 1990):

- nonaqueous phase liquids (NAPLs)
- contaminants dissolved in the groundwater.

Nonaqueous phase liquids are those that are not dissolved in groundwater. Nonaqueous phase liquids migrate as a separate liquid phase. Although NAPLs are not dissolved in groundwater, they are generally *not* insoluble. NAPLs have *limited* solubility. A NAPL can be one of two types:

- DNAPL - dense nonaqueous phase liquid
- LNAPL - light nonaqueous phase liquid.

DNAPLs are denser than water, and migrate to below the water table. The most common DNAPLs are chlorinated hydrocarbons. One of the most notorious DNAPLs is trichloroethene, also called trichloroethylene (TCE). TCE is a solvent used in dry cleaning and metal degreasing (Fetter 1992). The structure of TCE is shown in Fig. 3.1.

LNAPLs are less dense than water, and float on the water table. The most common LNAPLs are hydrocarbon fuels (Fetter 1992). Hydrocarbon fuels are a complex mixture of aromatic (containing a benzene ring) and aliphatic hydrocarbons.

Fig. 3.1 Structure of TCE (after Fetter 1992).

The potential for the release of NAPLs to the environment is quite large due to the large amounts that are produced and handled every year (Kueper and McWhorter 1991). As an example of the potential for spills, the USEPA estimates that there are 800,000 individual underground gasoline storage tanks in the United States (USEPA 1986). Many of these tanks leak, and have released LNAPLs into the subsurface. NAPLs have also leaked from waste disposal facilities (Kueper and McWhorter 1991).

NAPL releases to the subsurface are a problem for two main reasons. First, they result in 'secondary' groundwater contamination. The initial spill of a NAPL is not in itself a serious problem. If the NAPL was insoluble, it would only be a danger if a well was placed directly into a pool of NAPL. However, NAPLs do have limited solubility (Domenico and Schwartz 1990). NAPL solubility limits are often orders of magnitude greater than allowable drinking water standards (Kueper and McWhorter 1991). As a NAPL pool slowly dissolves into the groundwater, a plume of contaminated groundwater forms (Domenico and Schwartz 1990). The second major problem is the potential for explosions. If flammable NAPLs volatilize in the subsurface, the fumes can migrate into neighbouring buildings. The fumes may ignite and result in an explosion (Mercer and Cohen 1990).

This chapter provides a brief summary of NAPL contaminant transport. Initially, a simplified conceptual model for NAPL migration in the subsurface is discussed. A summary of the major parameters necessary to understand NAPL migration in the

subsurface is provided. With this understanding of NAPL behaviour, the conceptual contaminant migration model is reexamined in greater detail. Following this, NAPL transport modelling is discussed, and an example is provided.

3.2 CONCEPTUAL MIGRATION MODEL

3.2.1 LNAPL

An illustration of LNAPL migration in the subsurface is shown in Fig. 3.2. The LNAPL percolates down through the soil to the water table. Residual LNAPL is left on the soil through which it passes. Since the LNAPL is less dense than water, the pool floats on the water table. Some LNAPL slowly dissolves into the groundwater, forming a solute plume. If the LNAPL is volatile, it will volatilize into the soil gas.

3.2.2 DNAPL

An illustration of DNAPL migration in the subsurface is shown in Fig. 3.3. The DNAPL also percolates down through the soil, leaving residue behind. When the mobile DNAPL encounters the water table, it continues descending. The DNAPL sinks until it encounters an impermeable layer. The DNAPL pool slowly dissolves, creating a solute plume. Residual DNAPL in the unsaturated zone also volatilizes into the soil gas as does DNAPL solute in the groundwater.

Fig. 3.2 LNAPL migration in the subsurface (after Walther *et al.* 1986).

Fig. 3.3 DNAPL migration in the subsurface (after Walther *et al.* 1986).

3.3 IMPORTANT PARAMETERS

To understand the mechanics of NAPL migration, it is necessary to understand the various parameters involved. In this section, a brief review of the important parameters is provided. Following this, the conceptual migration model is considered in greater detail.

3.3.1 Saturation, S

For each fluid in a volume of soil, the saturation is the volume fraction of the voids occupied by that liquid. This can be expressed in equation form (Domenico and Schwartz 1990):

$$S_i = \frac{V_i}{V_{\text{VOIDS}}} \quad [3.1]$$

Where S_i = saturation of fluid i

V_i = volume of fluid i

V_{VOIDS} = volume of voids.

Thus, the saturation describes the relative abundance of each fluid (Domenico and Schwartz 1990).

The saturation of each fluid can vary between zero and one. The saturation of all the fluids sum to unity (Mercer and Cohen 1990). In the vadose zone (unsaturated zone) there can be three fluids: air, water, and NAPL. In the saturated zone there can be two fluids: water and NAPL.

3.3.2 Interfacial tension, σ

Interfacial tension is theoretically defined as the free surface energy at the interface between two immiscible or nearly immiscible fluids (Villaume 1985). This energy results from the difference between the mutual attraction of like molecules on each side of the interface, and the attraction of dissimilar molecules across the interface (Schowalter 1979). Surface tension is the interfacial tension between a fluid and air. Since a surface having free energy tries to contract, there is an effective tension at the interface (Bear 1979). A diagram of how interfacial tension acts in a soil pore or rock joint is shown in Fig. 3.4.

The interfacial tension between any two fluids is effectively a constant for those two fluids. For example, the interfacial tension between benzene and water is (Mercer and Cohen 1990):

$$\sigma = 36.5 \text{ dyne/cm}$$

The units of interfacial tension are energy per unit area. The value of the interfacial tension for two fluids decreases with increasing temperature. Changes in pH and the presence of surfactants also change the interfacial tension (Mercer and Cohen 1990).

3.3.3 Wettability

Wettability is a qualitative term that describes the preferential spreading of one fluid over a solid surface when more than one fluid is present (Mercer and Cchen 1990). Thus, it describes the tendency of one fluid to be attracted to a solid surface instead of another

Fig. 3.4 Interfacial tension in a soil pore or rock joint (after Bear 1979).

(Domenico and Schwartz 1990). When two fluids are present in soil or rock, the wetting fluid coats solid surfaces and occupies the smaller constrictions. The nonwetting fluid occupies the larger openings (Mercer and Cohen 1990). While the relationship as to which fluid is the wetting fluid depends on the solid surface and on the fluids involved, some generalizations are possible (Albertsen *et al.* 1986):

- On rock forming minerals, water is generally wetting with respect to NAPLs or air.
- On rock forming minerals, NAPLs are generally wetting when in contact with air, but nonwetting when in contact with water.
- On organic matter, NAPLs are generally wetting when in contact with air or water.

Demond and Roberts (1987) suggest that wettability can be measured by the contact angle, ϕ . This is the angle at the fluid solid interface. It can be measured by placing a droplet of water on a solid surface in a bath of NAPL as shown in Fig. 3.5. If the contact angle is less than 90° , then water is the wetting fluid. Otherwise, the NAPL is the wetting fluid (Domenico and Schwartz 1990). The contact angle can theoretically be calculated using Young's equation which considers static equilibrium of the three substance interface (Bear 1979):

$$\cos\phi = \frac{(\sigma_{ns} - \sigma_{ws})}{\sigma_{nw}} \quad [3.2]$$

Fig. 3.5 Measurement of contact angle, ϕ (after Demond and Roberts 1987).

Where σ_{ns} = NAPL - solid interfacial tension

σ_{ws} = water - solid interfacial tension

σ_{nw} = NAPL - water interfacial tension

3.3.4 Capillary pressure, P_c

Capillary pressure is defined as the difference in pressure across an interface. Using the terminology in Fig. 3.6, capillary pressure is (Mercer and Cohen 1990):

$$P_c = P_{nw} - P_w \quad [3.3]$$

Where P_c = capillary pressure

P_{nw} = pressure of nonwetting fluid

P_w = pressure of wetting fluid.

This pressure difference results from the interfacial tension which exists between the two fluids. For the rock joint or soil pore shown in Fig. 3.6, the interfacial tension acting at an angle of ϕ (contact angle) with the solid surface supports a pressure difference. The interface acts as a stretched membrane. Based on static equilibrium, the relationship between capillary pressure, interfacial tension, and contact angle can be derived. For a circular pore (tubular) (Bear 1979):

$$P_c = 2\sigma\cos\phi/r \quad [3.4]$$

Where r = radius of circular pore.

For a planar clay fissure or rock joint as shown in Fig. 3.7, the relationship is (Kueper and McWhorter 1991):

Fig. 3.6 Pressure across an interface (after Bear 1979).

Fig. 3.7 Planar clay fissure or rock joint (after Kueper and McWhorter 1991).

$$P_c = 2\sigma \cos\phi / e \quad [3.5]$$

Where e = fissure or joint opening.

In reality, however, the medium of interest is usually porous, not a circular tube or planar fracture. A close look at the wetting/nonwetting fluid interface in a porous medium is shown in Fig. 3.8. The radius of curvature of this interface is a function of saturation. Thus, the capillary pressure for this system is a function of saturation. An example of such a relationship is shown in Fig. 3.9. This relationship is identical to the matric potential used to describe unsaturated soil behaviour. The capillary pressure in Fig. 3.9 is negative because it is plotted with $P_c = P_w - P_{nw}$, which is the reverse of [3.3]. It is a matter of convention as to which is used. There is a theoretical equation describing the capillary pressure in a porous medium as a function of interface shape. Theoretically, the capillary pressure is (Leverett 1941):

$$P_c = \sigma \left(\frac{1}{R_1} + \frac{1}{R_2} \right) \quad [3.6]$$

Where R_1, R_2 = the radii of curvature of the interface as shown in Fig. 3.8.

As can be seen from Fig. 3.9, the capillary pressure/saturation relationship is a hysteretic one. The drainage curve (wetting fluid being expelled) lies above the imbibition curve (nonwetting fluid being expelled). If a soil is undergoing drainage, then starts to rewet, the capillary pressure crosses from the drainage to the imbibition curve by intermediate scanning curves (Fetter 1992). The hysteresis in this relationship is due to two reasons. First, there is the so-called ink-bottle effect (Fetter 1992). Consider the two systems

Fig. 3.8 a) Porous medium with two phases. b) Wetting/nonwetting fluid interface (after Leverett 1941).

Fig. 3.9 Capillary pressure as a function of wetting fluid saturation (after Fetter 1992).

Fig. 3.10 Ink bottle effect for a) drainage; b) imbibition (after Fetter 1992).

shown in Fig. 3.10. The capillary pressure in these two systems are the same since the radii of curvature are the same. However, the system which is about to undergo drainage (system a) has a saturated pore. The system which is undergoing imbibition (system b) does not have a saturated pore. The result is different saturations at the same capillary pressure. The second reason for hysteresis is that the contact angle is greater during imbibition than during drainage (Fetter 1992).

Another feature of Fig. 3.9 is the capillary pressure P_d , called the displacement pressure. This pressure is also called the threshold value, imbibition pressure, bubbling pressure, or air entry value (when air is the nonwetting phase). The pore is saturated at capillary pressures below the displacement pressure. The displacement pressure plays a fairly important role in contaminant transport. This is discussed in section 3.3.5.

Additional features of Fig. 3.9 are the residual saturation of the nonwetting fluid (S_{nwr}), and the irreducible wetting fluid saturation (S_{wr}) which is also known as the residual saturation of the wetting fluid (S_{wr}). The residual saturations are discussed in section 3.3.6.

It is capillary pressure that results in wetting fluids being drawn into porous media (Bear 1979). As a NAPL release descends in the vadose zone, the NAPL spreads horizontally by capillary action. This is only possible since NAPLs are usually the wetting fluid when in contact with air (de Pastrovich *et al.* 1979).

3.3.5 Displacement pressure, P_d

The capillary pressure that must be developed for a *nonwetting* NAPL to enter a water filled pore is called the displacement pressure. This is the usual case for a DNAPL in the saturated zone. Before a DNAPL can enter a pore, sufficient head must be developed to overcome the capillary force (Mercer and Cohen 1990). This effect is apparent in the capillary pressure-saturation relation in Fig. 3.9. The capillary pressure must exceed P_d before the water saturation is reduced below 1.0. Since [3.4] and [3.5] define conditions at static equilibrium, these equations also describe the entrance pressures. If the capillary pressure exceeds these entrance pressures, the result is a dynamic situation and the DNAPL-water interface moves into the soil pore, clay fissure or rock joint (Kueper and McWhorter 1991). Once the capillary forces are overcome, the oil filament enters the pore and displaces the water (Schowalter 1979). The entrance conditions are shown in Fig. 3.11.

The displacement pressure concept also allows one to calculate the depth to which a DNAPL disposal pond can be filled before DNAPL penetrates into the soil. Sufficient head must be developed to generate enough pressure to overcome [3.4] or [3.5]. The required pond depth for entry into a clay fissure at the bottom of the pond is (Kueper and McWhorter 1991):

Fig. 3.11 Entrance conditions for a soil pore, clay fissure, or rock joint (after Kueper and McWhorter 1991).

$$H_D = \frac{2\sigma}{(\rho_D - \rho_w)ge} \quad [3.7]$$

Where ρ_D = DNAPL density

ρ_w = density of water

g = acceleration due to gravity.

Similarly, [3.4] and [3.5] help in the prediction of where DNAPLs will migrate to in porous, or fractured, media. The DNAPL will migrate until the equilibrium conditions defined by [3.4] and [3.5] are achieved. Only pores or fractures with entry pressures less than the capillary pressure are invaded (Kueper and McWhorter 1991).

3.3.6 Residual saturation, S_r

Residual saturation is the saturation of a fluid at which the fluid is immobilized by capillary forces (Mercer and Cohen 1990). As such, the residual saturations are the end points of the capillary pressure/saturation curve shown in Fig. 3.9. The wetting phase can not be reduced below the irreducible wetting phase saturation (S_{wr}). This is also called the residual wetting fluid saturation (S_{wr}). During imbibition, the nonwetting phase can not be reduced below the residual saturation of the nonwetting fluid (S_{nwr}). The residual saturation of the nonwetting fluid (S_{nwr}) is equal to the distance from the imbibition curve endpoint to the 100% water saturation line in Fig. 3.9, since S_{nw} and S_w sum to 100%.

For a NAPL, the residual saturation is the saturation at which the NAPL becomes discontinuous. Two modes of residual saturation are pendular saturation and insular

Fig. 3.12 Two modes of residual saturation. a) Pendular saturation; b) Insular saturation (after Domenico and Schwartz 1990).

saturation. These are shown in Fig. 3.12. Since a NAPL is typically nonwetting in the saturated zone, it exists in insular saturation. However, in the unsaturated zone, a NAPL is the wetting fluid with respect to air and exists in pendular saturation (Domenico and Schwartz 1990).

Since water is the wetting phase in the saturated zone, it can be reduced to the irreducible wetting phase saturation (S_{wi}) if sufficient NAPL floods the pores. Since water is practically always the wetting fluid, it can only be found in pendular saturation.

Residual saturation of NAPLs results in a lasting source of contamination. Although NAPL solubility is limited, the solubility limits are often orders of magnitude greater than drinking water standards. NAPL at residual saturation in the vadose zone may get dissolved by infiltrating rainwater and get carried down to the saturated zone. Similarly, NAPL at residual saturation in the saturated zone is slowly dissolved by flowing groundwater. The result is a long term source of dissolved NAPL (Mercer and Cohen 1990).

The value of the residual saturation depends on (Mercer and Cohen 1990):

- pore size distribution
- wettability
- ratio of viscosities of pore fluids
- ratio of densities of pore fluids

- interfacial tension.

3.3.7 Relative permeability, k_r

When several fluids are present in a porous medium they restrict the flow of each other as they compete for space. This is quantified by the relative permeability of each fluid. The relative permeability of a fluid is the ratio of the effective permeability of the fluid at a given saturation to the intrinsic permeability (Mercer and Cohen 1990).

$$k_r = \frac{K_i}{K} \quad [3.8]$$

Where k_r = relative permeability

K_i = effective permeability for fluid i

K = intrinsic permeability (recall equation [2.3] in chapter 2)

The role of relative permeability is more evident in the Darcy's law equation for one dimensional two phase flow (Fetter 1992):

$$Q_w = \frac{-k_{rw} K \gamma_w}{\mu_w} A \frac{dh_w}{dl} \quad [3.9]$$

$$Q_{nw} = \frac{-k_{rnw} K \gamma_{nw}}{\mu_{nw}} A \frac{dh_{nw}}{dl} \quad [3.10]$$

Where Q_w, Q_{nw} = volume flow rate of wetting and nonwetting fluids

k_{rw}, k_{rnw} = relative permeabilities of wetting and nonwetting fluids at the appropriate saturation

K = intrinsic permeability of porous medium

γ_w, γ_{nw} = unit weight of wetting and nonwetting fluids

μ_w, μ_{nw} = dynamic viscosity of wetting and nonwetting fluids

A = cross-sectional area of flow

$dh_w/dl, dh_{nw}/dl$ = gradient of head of wetting and nonwetting fluids.

Relative permeability is a function of saturation. A typical relative permeability diagram is shown in Fig. 3.13. When either fluid has a saturation of 1.0 (ie - one phase flow), the relative permeability is 1.0. With increasing wetting fluid saturation, the nonwetting fluid occupies less pore volume and its relative permeability decreases. At a nonwetting fluid saturation greater than zero, the nonwetting fluid relative permeability drops to zero. The nonwetting fluid is immobilized. The nonwetting fluid is at residual saturation (S_{nwr}). The same process happens to the wetting fluid. Below the irreducible wetting fluid saturation, the wetting fluid is immobilized (Palmer and Johnson 1989). Thus, the endpoints of the relative permeability curve are the residual saturations, which are the same as for the capillary pressure curve.

The relative permeability curve also exhibits hysteresis. An example of this hysteresis is shown in the graphs in Fig. 3.14, which are the experimental relative permeability curves for the flow of TCE and water in a fine sand. Both the wetting (water) and nonwetting (TCE) fluids have different drainage and imbibition curves (Fetter 1992).

The concepts of relative permeability and multiphase flow also extend to three phase flow systems. An example of such a system is the movement of a NAPL down through the

Fig. 3.13 Typical relative permeability curve as a function of wetting fluid saturation
(after Fetter 1992).

Fig. 3.14 Hysteresis in relative permeability measurements. a) Water relative permeability; b) TCE relative permeability (after Lin *et al.* 1982).

unsaturated zone. Three phases are present: water, NAPL, and air. The generalized multiphase flow equation is (Demond and Roberts 1987):

$$q_i = - \frac{k_r K \gamma_i}{\mu_i} \nabla \left[\frac{P_i}{\gamma_i} + z \right] \quad [3.11]$$

Where q_i = flux vector of fluid i

k_r = relative permeability of fluid i at its saturation in the system

γ_i = unit weight of fluid i

μ_i = dynamic viscosity of fluid i

P_i = pressure of fluid i

z = elevation above a datum.

For such a system, it is necessary to know the relative permeability of each of the three fluids. Such data are typically presented in ternary diagrams as shown in Fig. 3.15. A ternary diagram can be plotted showing k_r (as a percent in Fig. 3.15) for each fluid. Some simplifications are made when attempting to model three phase systems (Faust 1985). The water relative permeability is usually assumed to be a function of water saturation only. This is not an unreasonable assumption. The ternary diagram for water in Fig. 3.15 shows that water relative permeability is a function of water saturation only. The air phase is often assumed to have atmospheric pressure in all locations. Consequently, air *flow* is neglected and the air relative permeability need not be calculated. The only relative permeability that needs to be calculated based on three phase flow is that of the nonaqueous phase. The ternary diagram for oil in Fig. 3.15 shows that k_m (nonaqueous phase relative permeability) is a function of all three

Fig. 3.15 Ternary diagram for three phase flow relative permeabilities (after Saraf 1967).

saturations (water, nonaqueous, and air). A simplified method of calculating k_m has been developed by petroleum engineers. This method calculates k_m for a three phase system using data from two two-phase flow systems (Fetter 1992):

1) water-nonaqueous phase system

2) air-nonaqueous phase system.

It is unnecessary to perform three phase relative permeability tests for this method. Two separate two phase flow tests can be done instead. The relative permeability of the nonaqueous phase in a three phase system is (Faust 1985):

$$k_{ra} = k^*_{rnw} \left[\left(\frac{k_{rnw}}{k^*_{rnw}} + k_{rw} \right) \left(\frac{k_{rna}}{k^*_{rnw}} + k_{ra} \right) - (k_{rw} + k_{ra}) \right] \quad [3.12]$$

Where k_m = relative permeability of nonaqueous phase in the three phase system

k_{mw} = relative permeability of nonaqueous phase in water-nonaqueous phase system as a function of water saturation (S_w)

k_{rw} = relative permeability of water in water-nonaqueous phase system as a function of water saturation (S_w)

k_{ma} = relative permeability of the nonaqueous phase in the air-nonaqueous phase system as a function of air saturation (S_a)

k_{ra} = relative permeability of air in air-nonaqueous phase system as a function of air saturation (S_a)

k^*_{mw} = relative permeability of the nonaqueous phase in the water-nonaqueous phase system at the residual saturation of water.

The use of relative permeabilities and Darcy's law for multiphase flow in modelling

multiphase flow will be briefly discussed in a later section. It will not be attempted to provide a complete description of multiphase flow modelling since the topic is beyond the scope of this report. Relative permeability *testing* is discussed in chapter 4.

3.3.8 Solubility

The aqueous solubility of a chemical is defined as the maximum concentration that will dissolve in pure water. When a NAPL is composed of several chemicals, the dissolved concentrations will be less than those of the pure components. For an NAPL composed of several chemicals, the more soluble components dissolve first. The remaining undissolved mixture is less soluble. This process is called weathering when describing the dissolution of gasoline spills (Mercer and Cohen 1990).

Using the concept of fugacity, Mackay *et al.* (1991) have suggested that the following simple equation describes the equilibrium concentration of each NAPL component dissolved in the surrounding groundwater:

$$y_i = K_i x_i \quad [3.13]$$

Where y_i = mole fraction of component i dissolved in the surrounding water (moles of component i / moles of water)

K_i = constant for component i

x_i = mole fraction of component i in the NAPL pool (moles of i / moles of NAPL).

Fairly complex models have been developed using this equation to describe the

dissolution of a multicomponent NAPL over time as it is passed by flowing groundwater. However, the actual dissolution depends on the contact area between the NAPL pool and the groundwater. This area is extremely difficult to calculate for a mass of NAPL at residual saturation. NAPL at residual saturation dissolves much more quickly than a pool of equivalent mass since it has such a high contact area with the flowing groundwater. It should be noted that for either a pool of NAPL, or NAPL at residual saturation, the rate of dissolution is slow. Consequently, dissolution is an ineffective removal process (Mercer and Cohen 1990).

3.3.9 Volatilization

Volatilization is the transfer of matter from the liquid phase to the gas phase. Volatilization is a more important process for NAPLs with high vapour pressures. Volatilization is also expected in sandy soils or where the NAPL is quite shallow. NAPLs can volatilize from NAPL at residual saturation in the vadose zone, from pools of LNAPL floating on the water table, or from NAPL in a dissolved plume (Mercer and Cohen 1990).

Volatilization is important because the soil gas can migrate and form an explosion hazard if the NAPL is flammable (Fussel *et al.* 1981). Flow of soil gas results from diffusion and density gradients. If soil gas formed from NAPL at residual saturation in the vadose zone is denser than air, it will sink to the water table. There, it will migrate along the water table and also dissolve into the groundwater (Mercer and Cohen 1990).

Modelling the volatilization of NAPLs into the unsaturated zone makes use of Raoult's law and Henry's law. Raoult's law is used to predict the partial pressure of NAPL in the soil gas due to volatilization from undissolved NAPL (Mercer and Cohen 1990). Volatilization can happen from residual LNAPL or DNAPL in the unsaturated zone. Volatilization can also happen from LNAPL pooled on the water table. Raoult's law is useful for predicting the volatilization of multicomponent NAPLs, such as gasoline. Raoult's law is:

$$P_A = X_A P_A^\circ \quad [3.14]$$

Where P_A = partial pressure of NAPL component A in the soil gas
 X_A = mole fraction of component A in the NAPL
 P_A° = vapour pressure of pure component A (a property of fluid A)

Henry's law is useful for predicting the volatilization of organics from aqueous solution (Mercer and Cohen 1990). If there is sufficient NAPL present that some reaches the water table, then a dilute solute plume will form. As this solute plume moves downgradient, organics can volatilize off into the soil gas. Henry's law is:

$$P_A = K_H C_A \quad [3.15]$$

Where P_A = partial pressure of the organic compound in the soil gas
 K_H = Henry's law constant for the organic compound (atm m³ / mol)
 C_A = concentration of the organic compound in the groundwater (mol / m³).

3.3.10 Unit weight, γ

The unit weight of a substance is the product of its density (ρ) and the acceleration due to gravity ($g = 9.81 \text{ m/s}^2$). The unit weight of a NAPL determines whether it is a DNAPL or an LNAPL. DNAPLs (NAPLs with specific gravities greater than one) can also be called sinkers. LNAPLs (NAPLs with specific gravities less than one) can also be called floaters (Mercer and Cohen 1990).

3.3.11 Viscosity, μ

Viscosity can qualitatively be described as the internal friction that resists flow. A low viscosity fluid flows more rapidly through the soil. Throughout this report, viscosity should be taken as meaning the dynamic viscosity. There is another type of viscosity called the kinematic viscosity (ν), which is equal to:

$$\nu = \frac{\mu}{\rho} \quad [3.16]$$

Where ν = kinematic viscosity

μ = dynamic viscosity

ρ = density.

Kinematic viscosity can be measured with ASTM test methods D445 and D446. The dynamic viscosity is then calculated using [3.16].

3.4 DETAILED CONCEPTUAL MIGRATION MODEL

3.4.1 LNAPL

A detailed schematic of LNAPL migration is shown in Fig. 3.16. As the LNAPL flows downward, it spreads horizontally by capillary action. LNAPL is left at residual (pendular) saturation in the vadose zone. If the spill is sufficiently large, LNAPL will reach the saturated zone. There, the LNAPL spreads along the capillary fringe and may cause the capillary fringe to collapse. The LNAPL can depress the groundwater table due to its weight. Since the solubility of the LNAPL is limited, it does not completely dissolve into the groundwater. Since the specific gravity is less than one, it floats on the water table. Some LNAPL will dissolve into the groundwater, forming a dissolved plume. If the LNAPL is volatile, it will volatilize from the pool on the water table, from the LNAPL at residual saturation in the vadose zone, and possibly from the dissolved plume. Since the LNAPL pool floats, it will tend to flow down gradient along the sloping water table (Feenstra and Cherry 1988; Mercer and Cohen 1990).

3.4.2 DNAPL

A detailed schematic of DNAPL migration is shown in Fig. 3.17. As the DNAPL descends through the vadose zone, it will leave DNAPL at residual (pendular) saturation. DNAPL will volatilize in this region. The dense vapours will then sink to the groundwater table. If the release of DNAPL is sufficiently large, mobile DNAPL will reach the saturated zone. Since it is denser than water (specific gravity greater than one), it will sink. As it sinks, it will leave DNAPL at residual (insular) saturation.

Fig. 3.16 LNAPL migration (after Pinder and Abriola 1986).

Fig. 3.17 DNAPL migration (after Feenstra and Cherry 1988).

DNAPL vapours, and DNAPL at residual saturation in the saturated zone, can dissolve into the groundwater resulting in a dissolved plume. If there is sufficient DNAPL, some may reach an impermeable layer as a mobile phase. An impermeable layer would be one where the displacement pressure of the pores exceeds the capillary pressure. DNAPL will then spread laterally. The result is a pool of DNAPL. If the DNAPL pool is on bedrock, it will tend to flow in the direction of the dip. This direction may be opposite to the direction of groundwater flow. Any pool of DNAPL will also release dissolved contaminants. DNAPL can also enter into joints in the bedrock if it can overcome the displacement pressure (Feenstra and Cherry 1988; Mercer and Cohen 1990).

For both the LNAPL and DNAPL, there is an additional problem. Percolating rain water can dissolve NAPL at residual saturation in the vadose zone. This process will carry more organic contaminants down to the saturated region, adding to the dissolved plume (Mercer and Cohen 1990).

3.5 MODELLING THE MIGRATION OF NONAQUEOUS PHASE LIQUIDS

NAPL migration is extremely difficult to model due to the nonlinearities involved (Faust 1985). This section will not attempt to describe the modelling strategies. However, it is desired to give the reader an understanding of the role of relative permeability in controlling the migration of nonaqueous phase liquids. The governing equations are provided as well as some simplifications suggested by Faust (1985). The solution of

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these equations by finite difference or finite element techniques will not be discussed.

The most complicated situation to model is the migration of a NAFL descending in the unsaturated zone. However, this is also the most general situation due to the presence of three phases. The continuity equations for three phase flow are (Peaceman 1977):

$$\nabla \left[\frac{K \rho_w k_{rw}}{\mu_w} (\nabla P_w - \rho_w g \nabla D) \right] + q'_w = \frac{\partial n \rho_w S_w}{\partial t} \quad [3.17]$$

$$\nabla \left[\frac{K \rho_n k_{rn}}{\mu_n} (\nabla P_n - \rho_n g \nabla D) \right] + q'_n = \frac{\partial n \rho_n S_n}{\partial t} \quad [3.18]$$

$$\nabla \left[\frac{K \rho_a k_{ra}}{\mu_a} (\nabla P_a - \rho_a g \nabla D) \right] + q'_a = \frac{\partial n \rho_a S_a}{\partial t} \quad [3.19]$$

Where k_{ri} = relative permeability of each phase i (water=w, nonaqueous=n, air=a)

D = depth

q'_i = mass source/sink term

n = porosity

There are 16 variables in these equations, but only three equations. The first, and most significant, assumption is that the air pressure is atmospheric. The continuity relationships then reduce to two equations. The densities and viscosities are assumed to be constant. The aquifer can also be considered incompressible. The remaining relationships are:

$$S_w + S_n + S_a = 1 \quad [3.20]$$

$$k_{rw} = f_1(S_w) \quad [3.21]$$

$$k_{rn} = f_2(S_w, S_a) \quad [3.22]$$

$$P_n - P_w = f_3(S_w) \quad [3.23]$$

$$P_n - P_a = P_n = f_4(S_a) \quad [3.24]$$

Equation [3.21] is a simplification by which the water relative permeability is a function of water saturation only. Equation [3.22] involves the use of [3.12] discussed earlier. Equation [3.23] assumes that the water-NAPL capillary pressure is a function of water saturation only. Similarly, equation [3.24] assumes that the nonaqueous-air capillary pressure (which is equal to the pressure in the nonaqueous fluid since the air is assumed to be at atmospheric pressure) is a function of air saturation only.

3.6 ANALYTICAL SOLUTION TO THE LINEAR WATER FLOOD

As mentioned in the previous section, modelling multiphase transport is very difficult. There is, however, a very simple multiphase flow problem for which an analytical solution exists. Buckley and Leverett (1942) developed a solution for the water saturation as a function of distance and time for a linear water flood. Buckley and Leverett were petroleum engineers interested in the ability of water to push oil out of sandstone reservoirs for collection. The mechanics of this problem are very similar to the infiltration of a NAPL into a soil by displacing water. This example is instructive because it shows the role of relative permeability in multiphase flow. Buckley and Leverett (1942) stated very clearly that the displacing fluid does not behave as a piston

ejecting the initial pore fluid. The displacing fluid (water) and the displaced fluid (oil) flow together and simultaneously through the pores. As mentioned in chapter 2, it is the fundamental basis of this research that permeability alone is an insufficient parameter for modelling NAPL transport through clay landfill liners or natural clayey soils. Engineers interested in the chemical interaction between clay soil and organic chemicals must also understand the nature of nonaqueous phase liquid transport. The following analytical solution to a very simplified problem shows that permeability alone is insufficient to solve this problem.

The problem considered here is the one dimensional two phase flow of water into an initially oil saturated sand. Capillary effects are ignored. It is specified that the water flux into the upgradient end of the system is constant with time. By continuity, the total flux (q_t) is the same throughout the system as shown in Fig. 3.18.

It is required to find the water saturation as a function of distance and time. For an element in the interior of this system, continuity of the water phase gives:

$$\left(\frac{\partial S_w}{\partial t}\right) = \frac{1}{n} \left(\frac{\partial q_w}{\partial x}\right) \quad [3.25]$$

Where q_w = water flux

S_w = water saturation

x = distance from inlet

n = porosity.

Fig. 3.18 Linear water flood boundary conditions.

Rearranging [3.25] gives:

$$\left(\frac{\partial x}{\partial t}\right)_{S_w} = \frac{1}{n} \left(\frac{\partial q_w}{\partial S_w}\right)_t \quad [3.26]$$

This states that the rate of advance of a plane of a given water saturation is proportional to the derivative of the water flux with respect to water saturation. However, the total water flux:

$$q_w = q_t f_w \quad [3.27]$$

Where f_w = fraction of total flux (q_t) which is water flux (q_w).

In other words, f_w is the fraction of water in the flowing phase as a function of water saturation. It is important to note that the fraction of water flowing (f_w) is not equal to the water saturation (S_w), but is a function of S_w as will be shown later. Since the total flux (q_t) for this problem is constant, then:

$$\left(\frac{\partial x}{\partial t}\right)_{S_w} = \frac{q_t}{n} \left(\frac{\partial f_w}{\partial S_w}\right)_t \quad [3.28]$$

Since $\partial x/\partial t$ is constant with time, then

$$x = \frac{q_t}{n} \left(\frac{\partial f_w}{\partial S_w}\right)_t t \quad [3.29]$$

But, by the definition of f_w

$$f_w = \frac{q_w}{q_w + q_o} \quad [3.30]$$

Where q_o = oil flux

and

$$q_w = - \frac{K k_{rw} \gamma_w}{\mu_w} \frac{dh_w}{dx} \quad [3.31]$$

$$q_o = - \frac{K k_{ro} \gamma_o}{\mu_o} \frac{dh_o}{dx} \quad [3.32]$$

If we assume that $\gamma_w \approx \gamma_o$, and that the gradients in the two phases are roughly equal, then substitution of [3.31] and [3.32] into [3.30] gives:

$$f_w = \frac{1}{1 + \frac{k_{ro} \mu_w}{k_{rw} \mu_o}} \quad [3.33]$$

It can be seen from [3.33] that f_w does not equal S_w , but is a function of S_w through the relative permeabilities. The following procedure is then used to calculate the saturation distribution for the following parameters:

$$q_t = 0.25 \text{ m/day}$$

$$n = 0.25$$

$$k_{rw} = \frac{(S_w - 0.16)^2}{0.64} \quad [3.34]$$

$$k_{ro} = \frac{(0.8 - S_w)^2}{0.64} \quad [3.35]$$

Fig. 3.19 Relative permeability relationships.

Fig. 3.20 f_w and df_w/dS_w .

Except that

$$k_{rw} = 0.0, k_{ro} = 1.0 \text{ for } S_w < 0.16$$

$$k_{rw} = 1.0, k_{ro} = 0.0 \text{ for } S_w > 0.80$$

These relative permeability relationships were used by Faust (1985) and are shown in Fig. 3.19. The parameters f_w and df_w/dS_w are calculated using k_{rw} , k_{ro} , μ_w , and μ_o and are shown in Fig. 3.20. The saturation distribution is calculated by:

- 1) Choosing a value of S_w
- 2) Finding df_w/dS_w for that saturation from [3.33].
- 3) Calculating the distance (x) from the inlet to the plane with the chosen saturation using [3.29]
- 4) Repeating for multiple values of S_w .

The resulting saturation distribution is multiple valued since the preceding calculation does not consider the total volume of water which has flooded the system. A mass balance must be done to find the distance (x_o) to the wetting front. This distance can be calculated from the following mass balance:

$$\int_0^{x_o} S_w dx = \frac{q_i}{n} t \quad [3.36]$$

The saturation distribution is then truncated at a distance x_o . The saturation distributions for $t = 1$ day and $t = 10$ days are shown in Fig. 3.21 and Fig. 3.22 respectively.

Capillary effects have been neglected in this solution. Capillary effects would result in a more gradual saturation transition, smoothing out the discontinuities (Buckley and

Fig. 3.21 Water saturation distribution at $t = 1$ day.

Fig. 3.22 Water saturation distribution at $t = 10$ days.

Leverett 1942).

The important conclusion to be drawn from this exercise is that permeability alone is not sufficient to model even this simple, one dimensional two phase flow problem. The displacing fluid does not flow as a piston, expelling the original fluid. Relative permeabilities must be used.

CHAPTER 4

RELATIVE PERMEABILITY TESTING

4.1 INTRODUCTION

Relative permeability, as discussed in chapter 3, is an important parameter for describing multiphase flow. This chapter is concerned with laboratory testing for the determination of relative permeability as a function of fluid saturation. The object of these tests is to obtain a curve of the form shown in Fig. 3.13. There are three means by which relative permeability can be obtained in the laboratory (Amyx *et al.* 1960):

- Direct measurement using steady-state testing
- Measurement using unsteady-state (transient) testing
- Calculation of relative permeability from laboratory capillary pressure data.

Only the first method will be discussed in detail. The object of this research is to obtain the relative permeability function of a soil for which clay-organic chemical interaction may be occurring. Chemical interaction is often a kinetic effect. Time may be required to reach equilibrium. A steady-state test, during which several pore volumes have flowed through the specimen, allows for chemical interaction to occur. Transient tests may proceed too quickly for chemical interaction to occur. The theory upon which the third method is based (calculation from capillary pressure data) does not account for chemical interaction. However, the second and third methods will briefly be discussed for completeness.

Relative permeability testing has been a subject of research among petroleum engineers since the 1930's. Standard test methods have existed for many years. Relative permeability testing has the objective of finding the relative permeability as a function of saturation. This procedure requires the accurate measurement of the relative permeability and of the fluid saturations. The major complication is that laboratory tests have to account for "end effects." These end effects result in saturation gradients and non constant pressure gradients. The different steady state tests differ in how they account for these end effects. To understand the reasoning behind these different test methods, it is important to understand the two types of end effects.

4.2 OUTLET END EFFECTS IN STEADY STATE RELATIVE PERMEABILITY TESTS

Relative permeability tests are typically conducted on cylindrical soil specimens or rock cores. A simplistic relative permeability test arrangement is shown in Fig. 4.1. Both oil and water enter at the inlet. Some method is used to mix the two fluids before they enter the specimen to ensure a uniform fluid saturation across the cross section. One dimensional flow should result. The water flux (q_w), oil flux (q_o), and the pressure drop (ΔP_w) are measured.

At the outlet end of the core, the two fluids exit the core into an open space and are removed. This outlet condition results in the outlet end effect. The two fluids must pass from a region of finite capillary pressure to one of zero capillary pressure (Richardson *et*

Fig. 4.1 Simple relative permeability test apparatus (after Thomsen 1987).

al. 1952). In the central portion of the core, the oil and water phases occupy the same void space. As discussed in chapter 3, interfacial tension at the water-oil interface supports a pressure difference between the two fluids. The pressure difference is a function of the saturations of the two fluids. However, the fluids exit into an open reservoir. In this reservoir, the two fluids must be at the same pressure. The capillary pressure is zero. The transition from finite capillary pressure to zero capillary pressure results in the outlet end effect.

The capillary pressure transition requires zero capillary pressure just inside the outlet face of the porous medium (Thomsen 1987). The process is easiest to visualize for an imbibition test where the specimen is initially oil saturated. Both water and oil are then injected into the inlet. When the water first reaches the outflow end of the core, it can not exit because the water pressure is less than the pressure in the oil filled space outside the core. Water then accumulates at the outflow face (Perkins 1957). With reference to Fig. 3.9 in chapter 3, it is apparent that for an imbibition process the capillary pressure is zero at the residual saturation of the nonwetting fluid (S_{nwr}). The wetting fluid (water) accumulates at the outflow face until:

$$S_o = S_{nwr} \quad [4.1]$$

$$S_w = 1 - S_{nwr} \quad [4.2]$$

$$P_c = P_o - P_w = 0 \quad [4.3]$$

Where S_o = oil saturation

S_{nwr} = residual nonwetting fluid (oil) saturation

S_w = water saturation

P_c = capillary pressure

P_o = pressure in oil phase

P_w = pressure in water phase

An analytical solution exists for the steady state saturation distribution for this test using the boundary condition from [4.1], [4.2], and [4.3] (Richardson *et al.* 1952). From Darcy's law for multiphase flow (equation [3.9] in chapter 3), the water flux for a system where elevation gradients are negligible is:

$$q_w = - \frac{k_{rw} K \gamma_w}{\mu_w} \frac{dP_w/\gamma_w}{dz} \quad [4.4]$$

Where z = distance from the inlet

Rearranging, and recognizing that the unit weight (γ_w) is constant

$$\frac{dP_w}{dz} = - \frac{q_w \mu_w}{K k_{rw}} \quad [4.5]$$

Similarly, for the oil phase

$$\frac{dP_o}{dz} = - \frac{q_o \mu_o}{K k_{ro}} \quad [4.6]$$

The capillary pressure is

$$P_c = P_o - P_w \quad [4.7]$$

Taking the derivative of [4.7] with respect to z , and substituting for [4.5] and [4.6]:

$$\frac{dP_c}{dz} = \frac{1}{K} \left(\frac{q_w \mu_w}{k_{rw}} - \frac{q_o \mu_o}{k_{ro}} \right) \quad [4.8]$$

By the chain rule

$$\frac{dP_c}{dz} = \frac{dP_c}{dS_w} \frac{dS_w}{dz} \quad [4.9]$$

Substituting [4.9] into [4.8] gives

$$\frac{dS_w}{dz} = \frac{1}{K} \left(\frac{q_w \mu_w}{k_{rw}} - \frac{q_o \mu_o}{k_{ro}} \right) \frac{1}{\frac{dP_c}{dS_w}} \quad [4.10]$$

All of the components on the right hand side of [4.10] are functions of S_w . Thus, [4.10] can be integrated to give the distance to a plane of given water saturation (S_w) using the boundary condition from [4.2] (Thomsen 1987):

$$z = L - \int_{S_w}^{1-S_w} \frac{\frac{dP_c}{dS_w}}{\frac{1}{K} \left(\frac{q_w \mu_w}{k_{rw}} - \frac{q_o \mu_o}{k_{ro}} \right)} dS_w \quad [4.11]$$

Where z = distance from inlet to the plane with water saturation S_w

L = specimen length

The solution to [4.11] requires:

- q_w, q_o as inputs
- evaluation of k_{rw} and k_{ro} as functions of saturation (Fig. 3.13)
- evaluation of dP_c/dS_w as a function of saturation (differentiation of the relationship shown in Fig. 3.9).

An example of a typical saturation distribution is shown in Fig. 4.2. The pressure in each phase can be calculated by integrating Darcy's law for each phase ([4.5] and [4.6]) back from a known outlet pressure. A typical pressure distribution is shown in Fig. 4.3.

There are two very important conclusions to be drawn from this analysis:

- The pressure gradient is only constant away from the outlet.
- The saturation is only constant away from the outlet.

In any region where the saturation is not constant, the capillary pressure is also not constant since capillary pressure is a function of saturation. If the capillary pressure is not constant, then the pressure (or head) *gradient* is not constant.

Relative permeability measurements are based on the average gradient in a section. For example, consider a test using the apparatus in Fig. 4.1, where K , μ_w , and γ_w are known. The water flux (q_w) is measured, as is the pressure drop (ΔP). The water relative permeability for that test is then calculated using Darcy's law for multiphase flow:

$$k_{rw} = \frac{q_w \mu_w}{K} \frac{1}{\Delta P / \Delta z} \quad [4.12]$$

Where Δz = test length over which the pressure drop is measured

The average gradient ($\Delta P / \Delta z$) is used. The calculation is only accurate if the gradient is constant over the test length.

Another important consideration is that the pressure drop is usually only measured in one

Fig. 4.2 Typical wetting phase saturation distribution as a function of distance from inlet
(after Thomsen 1987).

Fig. 4.3 Typical water and oil pressure distribution as a function of distance from the inlet
(after Thomsen 1987).

fluid. The pressure drop is typically measured only in the nonwetting phase (Geffen *et al.* 1951). Special semipermeable pressure pads are required to measure the wetting phase pressure (Richardson *et al.* 1952). It is conventional practice to attribute the pressure drop in the nonwetting phase to both phases (Geffen *et al.* 1951). The relative permeability to each fluid (k_{rw} , k_{ro}) is calculated using the same pressure drop. This assumption is only correct if the saturation is constant over the test length, as shown in Fig. 4.2 and Fig. 4.3.

In summary, the use of average gradients and the measurement of only nonwetting phase pressure make it essential to measure the pressure drop over a region of uniform saturation. It is incorrect to calculate relative permeabilities using inlet and outlet pressures only (Richardson *et al.* 1952). The pressure difference between the inlet and outlet does not reflect the true gradient in either phase. However, some test methods do use terminal pressures. For these tests, the calculated relative permeability depends on the gradient (or flow rate) used during the test. The distance over which the end effect has influence (the so-called exit length) is less for higher flow rates. This is shown in Fig. 4.4. Tests which use terminal pressures must use high flow rates to reduce the significance of end effects. The flow rates typically exceed *in situ* flow rates (Thomsen 1987). Gradients in the order of 400 may be necessary (Amyx *et al.* 1960).

The final problem posed by the outlet end effect is that it makes saturation measurement difficult (Geffen *et al.* 1951). To develop the relative permeability function shown in Fig.

Fig. 4.4 Wetting phase saturations for different flow rates (after Thomsen 1987).

Note: SCC/HR = standard cubic centimetres per hour

3.13, each relative permeability measurement has to be associated with a measured fluid saturation. The average fluid saturation over the test section is used. Saturation measurement is discussed later. If the saturation is not constant over the test section, the average saturation will not accurately reflect the true saturation. In other words, a relative permeability and a saturation can be determined using the average pressure drop and the average saturation. However, the point defined by these two values may not lie on the actual relative permeability curve. Geffen *et al.* (1951) reported that the actual relative permeability at the measured saturation could be up to 25% lower than the true value, due to the combined error in the relative permeability and the fluid saturation.

Relative permeability measurements must be done over a length of the specimen where:

- The fluid saturations are constant
- The capillary pressure is constant giving equal pressure drops in the two phases
- The pressure (or head) gradient is constant.

4.3 INLET END EFFECTS IN STEADY STATE RELATIVE PERMEABILITY TESTS

At the inlet, two fluids are forced into the core. However, it is difficult to distribute the two fluids uniformly over the entrance face (Thomsen 1987). The wetting fluid (water) is quickly imbibed wherever it first encounters the inlet face. The result is entry of the wetting fluid over a localized area (Kyte and Rapaport 1958). Once the wetting fluid enters the core, it is spread out across the cross section by capillary forces. However, a

certain inlet length is required to obtain one dimensional flow (Thomsen 1987). The inlet end effect is a two dimensional problem, in contrast to the outlet end effect which is a one dimensional phenomenon (Thomsen 1987). There are two ways of reducing the impact of the inlet end effect (Kyte and Rapoport 1958). The first method is to use a long specimen so that measurements can be taken at a sufficient distance from the inlet. Measurements can then be made in a region of one dimensional flow. The second method is to use a low fluid injection rate. The low flow rate results in lower pressure gradients. Capillary pressure gradients are then more effective in spreading out the fluids. The result is a reduced entrance length (Rapoport 1958).

4.4 STEADY STATE METHODS FOR RELATIVE PERMEABILITY TESTING

There are basically four standard methods for steady state relative permeability tests (Amyx *et al.* 1960):

- Penn State method
- Hafford method
- Dispersed-feed method
- Hassler method.

All four test methods have common elements. The test specimens are typically small rock core samples from petroleum reservoirs. The specimen is mounted in lucite, or in a pressurized rubber sleeve. The two (or three) phases are introduced simultaneously at the inlet through separate pipes. The two fluids are injected at a constant ratio until the fluids are produced at the outlet at the same ratio (Amyx *et al.* 1960). The relative

permeability is calculated using the measured flow rates and pressure drop. The saturation must then be measured using one of three methods.

The first method for saturation determination uses the electrical resistivity of the core as a measure of fluid saturation (Amyx *et al.* 1960). This method is convenient because the core need not be removed from the apparatus to make the measurement. However, a calibration curve is needed to convert the electrical resistivity to fluid saturation (Leverett 1939).

The second method is gravimetric. The core is removed from the apparatus and weighed. This technique is only applicable if one of the fluids is a gas. The liquid saturation can then be calculated using the porosity and unit weight (Amyx *et al.* 1960).

The third method for the determination of fluid saturation uses a volumetric balance between fluid input and fluid output to calculate the relative amounts of the fluids remaining in the specimen (Amyx *et al.* 1960).

The entire procedure is repeated at several different ratios of wetting to nonwetting fluid injection rates. Each test gives a wetting fluid relative permeability, a nonwetting fluid relative permeability, and the fluid saturations. The relative permeability curve is found from the different injection ratio tests.

The Penn State method is shown in Fig. 4.5. The specimen is mounted between porous mixing and end sections with similar properties to the test specimen. The intent is that the inlet effect will be confined to the mixing section, and the outlet effect will be confined to the end section. The test section should have uniform saturation and constant pressure gradients. Fluid saturations are determined by electrical resistivity measurements (Honarpour *et al.* 1986).

The Hafford method apparatus is shown in Fig. 4.6. This method uses a high flow rate to eliminate outlet end effects (Amyx *et al.* 1960). The nonwetting fluid is introduced through a porous disc. The wetting fluid is introduced through a semipermeable disc that allows only it to pass. The central portion of the inlet disc contains a pressure pad to measure the wetting fluid pressure (Richardson *et al.* 1952).

The dispersed-feed method apparatus is shown in Fig. 4.7. This method attempts to eliminate the inlet end effect by ensuring one dimensional flow. The wetting fluid is introduced through the dispersing section. The nonwetting fluid is introduced through radial grooves in the *outflow* face of the dispersing section (Richardson *et al.* 1952). The outlet end effect is reduced by using high flow rates (Amyx *et al.* 1960).

The Hassler method apparatus is shown in Fig. 4.8. Semipermeable membranes are used at each end of the specimen to keep the fluids separate at the inlet and outlet. For each wetting phase flow rate, the nonwetting phase flow rate is adjusted until the pressure

Fig. 4.5 Penn State apparatus (after Geffen *et al.* 1951).

Fig. 4.6 Hafford method apparatus (after Richardson *et al.* 1952).

Fig. 4.7 Dispersed-feed apparatus (after Richardson *et al.* 1952).

Fig. 4.8 Hassler method apparatus (after Osoba *et al.* 1951).

gradients are equal in the two phases. The two fluids can exit to different pressures due to the semipermeable discs. This process eliminates saturation gradients and the outlet end effect (Honarpour *et al.* 1986).

4.5 ALTERNATIVE METHODS FOR RELATIVE PERMEABILITY TESTING

There are two other ways of determining relative permeability in the laboratory. Although neither method is used in this research, they will be briefly discussed for completeness.

4.5.1 Unsteady-state (transient) tests

Unsteady-state tests are the most common for petroleum reservoir engineering applications. These tests have the advantage of being much faster than steady-state tests. However, the disadvantage is that the mathematical analysis is more complicated (Honarpour *et al.* 1986). In addition, the test does not provide relative permeability values. Instead, relative permeability ratios ($k_{r_{nw}}/k_{r_w}$) are obtained. Since relative permeability ratios are insufficient in this research, this test method can not be used.

Unsteady-state methods involve injecting a nonwetting fluid into a wetting fluid saturated specimen. The relative permeability ratio can then be calculated from:

- the pressure drop from the inlet to the outlet,
- the cumulative nonwetting fluid injected as a function of time,
- the cumulative wetting fluid produced at the outlet as a function of time, and

- the porosity.

4.5.2 Calculation from laboratory capillary pressure data

This method is based on a separate theory, whereby the one phase permeability of a soil or porous rock can be calculated from air-water capillary pressure data. This theory is based on the idealization of a porous medium as a bundle of capillary tubes of random radii. This theory states that the permeability can be calculated by integrating $1/(P_c)^2$ from the unsaturated capillary pressure-saturation relationship (Amyx *et al.* 1960). The intrinsic permeability is

$$K = \frac{(\sigma \cos \phi)^2}{2} n \lambda \int_{S=0}^{1.0} \frac{dS}{(P_c)^2} \quad [4.13]$$

Where K = intrinsic permeability

σ = interfacial tension

ϕ = contact angle

n = porosity

P_c = capillary pressure

S = saturation

λ = lithology factor to account for natural soil behaviour

This theory is then extended to two phase flow to obtain the relative permeability (k_{rw}) at the wetting phase saturation S_w :

$$k_{rw} = \frac{K_w}{K} = \frac{\int_{S_w=0}^{S_w} \frac{dS_w}{(P_c)^2}}{\int_{S_w=0}^{1.0} \frac{dS_w}{(P_c)^2}} \quad [4.14]$$

Where k_{rw} = wetting fluid relative permeability at saturation S_w

K_w = effective permeability to wetting phase

K = intrinsic permeability

S_w = wetting fluid saturation

Similarly, for the nonwetting phase

$$k_{rnw} = \frac{\int_{S_w}^{S_w=1.0} \frac{dS_w}{(P_c)^2}}{\int_{S_w=0}^{1.0} \frac{dS_w}{(P_c)^2}} \quad [4.15]$$

Where k_{rnw} = nonwetting fluid relative permeability

These are the simplest expressions available for relative permeabilities. Much more complicated versions exist. However, it is obvious that these expressions can not take into account chemical interactions which change the intrinsic permeability. For this reason, a steady state method is necessary for this research.

CHAPTER 5

ANALYTICAL MODELLING

5.1 INTRODUCTION

The second stage of this research involved the design and production of an apparatus to measure the relative permeability for the flow of an organic chemical and water through a clayey soil. The design of the apparatus required the estimation of:

- the pressure drop that would be measured
- the saturation distribution which would develop due to the outlet end effect.

Analytical modelling of the system was undertaken using assumed material properties. This chapter summarizes the results of this modelling exercise. Many of the material properties used for this analysis were quite different from the values for the soil that was used during the testing program. However, the results of this analysis are useful for understanding the design of the test apparatus, as presented in the following chapter.

5.2 ESTIMATION OF PRESSURE DROP

It was necessary to estimate the pressure drop that would be measured in order that the correct differential pressure transducer could be obtained. The pressure drop is a function of:

- distance over which measurement is made
- flow rate of both phases
- intrinsic permeability

- phase saturations, since these define the relative permeabilities of each fluid.

In addition, the flow rate had to be selected so that the test would occur within a reasonable length of time. Due to the number of parameters involved, it was necessary to estimate the pressure drop using several combinations of intrinsic permeability and relative permeability.

The intrinsic permeability was selected to have a maximum value of $1 \times 10^{-16} \text{ m}^2$, and a minimum value of $1 \times 10^{-17} \text{ m}^2$. This corresponds to a hydraulic conductivity between $1 \times 10^{-7} \text{ cm/s}$, and $1 \times 10^{-8} \text{ cm/s}$. The upper limit of $1 \times 10^{-7} \text{ cm/s}$ was selected since this is typically the maximum allowable hydraulic conductivity for clay landfill liners. The hydraulic conductivity of a soil is highly variable. Two identical soils, compacted in the same manner, can easily have hydraulic conductivities differing by an order of magnitude, hence the lower limit of $1 \times 10^{-8} \text{ cm/s}$. One problem with permeability testing is that it is generally desirable to operate in a predetermined range of hydraulic gradients. If the intrinsic permeability varies within an order of magnitude, so does the length of time to run the test. One test may take two days to run, and the next may take 20 days. It is very undesirable to have such uncertainty in the test length, but it is unavoidable.

Three relative permeability curves were used. These curves were selected to simulate a range of possible values that might be measured during the testing. The first curve is shown in Fig. 5.1. In the figure, subscript "g" refers to the nonwetting fluid (gasoline) and subscript "w" refers to water. The curve in Fig. 5.1 is representative of a soil in

Fig. 5.1 Relative permeability curve 1.

which two phase flow occurs over a wide range of saturations ($0.2 \leq S_w \leq 0.8$) and is completely symmetrical. The second curve is shown in Fig. 5.2. For this curve, two phase flow occurs in a limited range of saturations, all at high water saturations. The third curve is shown in Fig. 5.3. For this curve, two phase flow occurs in a limited range of saturations, all at low water saturations. The relative permeabilities are also quite low.

The two fluids initially selected were water and gasoline. Gasoline was selected since it is relatively inexpensive, and a common groundwater contaminant. During the actual testing phase, gasoline was not used. This decision will be discussed in the next chapter. However, the design was based on the properties of water and gasoline:

$$\text{Water viscosity} = \mu_w = 9.815 \times 10^{-4} \text{ Ns/m}^2$$

$$\text{gasoline viscosity} = \mu_g = 4.5 \times 10^{-4} \text{ Ns/m}^2$$

The distance over which the pressure drop was measured was 25 mm.

The analysis for the pressure drop assumes that the measurement will be made in a region of uniform saturation, and constant capillary pressure. Based on the outlet end effect discussion in chapter four, the water saturation is that at an infinite distance from the outlet:

$$S_w = S_{w\infty} \quad [5.1]$$

This water saturation occurs at a sufficient distance from the outlet that

Fig. 5.2 Relative permeability curve 2.

Fig. 5.3 Relative permeability curve 3.

$$\frac{dS_w}{dz} \approx 0 \quad [5.2]$$

The outlet end effect analysis presented in chapter four can be used to solve for S_{wm} . In chapter four, Darcy's law for multiphase flow for each phase was substituted into the expression for the derivative of the capillary pressure with respect to distance. Then, using the chain rule, [4.10] was derived. Using the subscripts for gasoline and water, [4.10] becomes:

$$\frac{dS_w}{dz} = \frac{1}{K} \left(\frac{q_w \mu_w}{k_{rw}} - \frac{q_g \mu_g}{k_{rg}} \right) \frac{1}{\frac{dP_c}{dS_w}} \quad [5.3]$$

Using the condition from [5.2]:

$$\frac{q_w \mu_w}{k_{rw}} - \frac{q_g \mu_g}{k_{rg}} = 0 \quad [5.4]$$

at an infinite distance from the outlet. This analysis requires sufficiently large pressure gradients that the elevation gradients are negligible. The two fluxes (q_w , q_g) and the two viscosities are input values. Thus, the water saturation far from the outlet is that where:

$$\frac{k_{rw}}{k_{rg}} = \frac{q_w \mu_w}{q_g \mu_g} \quad [5.5]$$

The procedure for calculating the pressure drop over a 25 mm test length in the region of uniform saturation is as follows:

Step 1: Select the water and gasoline fluxes.

Step 2: Calculate the relative permeability ratio in the region of uniform saturation from [5.5].

Step 3: Using the appropriate relative permeability relationship, find the unique water saturation where k_{rw}/k_{rg} is the value from step 2. Since linear hypothetical relative permeability relationships were chosen, this can be done by substitution of the relative permeability equations into [5.5]. For example, for curve 1:

$$k_{rw} = -0.3 + 1.5 S_w \quad \text{For } 0.2 \leq S_w \leq 0.8 \quad [5.6]$$

$$k_{rg} = 1.2 - 1.5 S_w \quad \text{For } 0.2 \leq S_w \leq 0.8 \quad [5.7]$$

Let

$$R = \frac{k_{rw}}{k_{rg}} = \frac{q_w \mu_w}{q_g \mu_g} \quad [5.8]$$

The value of R can be calculated from the input values. Substitution of [5.6] and [5.7] into [5.8] yields:

$$R = \frac{-0.3 + 1.5 S_w}{1.2 - 1.5 S_w} \quad [5.9]$$

Rearranging [5.9] gives:

$$S_w = S_{wm} = \frac{1.2 R + 0.3}{1.5 + 1.5 R} \quad [5.10]$$

Step 4: Calculate the relative permeabilities (k_{rw} , k_{rg}) in the region of uniform

saturation by substituting S_{wm} into the relative permeability functions.

Step 5: Calculate the pressure gradient in each phase using Darcy's law:

$$\frac{dP_w}{dz} = - \frac{q_w \mu_w}{K k_{rw}} \quad [5.11]$$

$$\frac{dP_g}{dz} = - \frac{q_g \mu_g}{K k_{rg}} \quad [5.12]$$

These two gradients should be equal as long as the elevation gradients are negligible in comparison to the pressure gradients. The elevation gradient is equal to unity. A pressure head gradient, $d(P/\gamma)/dz$, of 100 would be sufficient for the elevation gradient to be negligible since it is less than one percent of the hydraulic gradient.

Step 6: Calculate the pressure drop in the nonwetting phase (gasoline) by:

$$\Delta P = \frac{dP_g}{dz} (0.025m) \quad [5.13]$$

This should be the pressure drop in both phases.

This procedure was used to calculate the pressure drop for combinations of:

- both intrinsic permeabilities ($1 \times 10^{-16} \text{ m}^2$ and $1 \times 10^{-17} \text{ m}^2$)
- all three relative permeability curves
- various water and gasoline fluxes.

During an actual test, the relative permeability curve is obtained by varying the ratio of water flux to gasoline flux. Thus, the theoretical pressure drops were calculated for various ratios with the same total flux. The total flux which yielded the most reasonable test time without inducing excessively large gradients was

$$q_t = q_w + q_g = 1.12 \times 10^{-3} \text{ cm/min} \quad [5.14]$$

This was one of the total fluxes that could be selected with the pump. The pump will be described in the following chapter. This total flux was selected as the one which would result in a test time of approximately 3.2 days. The test time was based on the desire to permeate at least four pore volumes through the 25 mm long test section. A 15 mm inlet distance was assumed between the inlet and the upgradient pressure transducer. This distance would help to insure one dimensional flow through the test section, thus avoiding the inlet end effect. This distance is quite short, but the low flow rate should limit the inlet length to a small value. As well, the sample length had to be minimized due to space restrictions. Following sample saturation, this inlet distance would have to be flushed before the organic would permeate the test section. A void ratio of 0.8 was assumed. Based on these assumptions, the total volume (V) required per unit cross sectional area of the specimen was:

$$\begin{aligned} V &= 4(2.5\text{cm}) \frac{0.8}{1 + 0.8} + (1.5\text{cm}) \frac{0.8}{1 + 0.8} \\ &= 5.11 \text{ cm}^3/\text{unit cross sectional area} \end{aligned} \quad [5.15]$$

The test time based on a flux of 1.12×10^{-3} cm/min is 3.2 days. This would be the time following sample saturation to obtain a single point on the relative permeability curve.

The results of these calculations are shown in Fig. 5.4. This is the theoretical pressure drop as function of $q_w:q_g$ for:

Fig. 5.4 Pressure drop for a total flux of 1.12×10^{-3} cm/min.

- the same total flux (1.12×10^{-3} cm/min),
- the two intrinsic permeabilities, and
- the three relative permeability curves.

The flux ratios are those which the pump can supply. Ratios of 0:8 and 8:0 are one phase flow of gasoline and water respectively. Identical curves for other possible total fluxes (q_t) can be generated by multiplying the pressures by the ratio of the total flux to 1.12×10^{-3} cm/min. For example, a reduction in the total flux by an order of magnitude would reduce the pressures by one order of magnitude.

The unfortunate conclusion to be drawn from Fig. 5.4 is that the pressure drop is quite sensitive to the intrinsic permeability (or hydraulic conductivity as shown in the figure). Few pressure transducers are accurate over several orders of magnitude of pressure variation. The total flux must be varied for each test to ensure that the measured pressure is within the limits of the transducer.

It is also interesting to note that the pressure drop is not as sensitive to the shape of the relative permeability curve. The increasing pressure drop with increasing water flux is due to the higher viscosity of water. The sudden decrease in pressure drop at $q_w:q_g = 8:0$ for curve 3 is explained as follows. The relative permeabilities in curve 3 are all fairly small. For this soil, the fluids interfere with each other substantially. As a result, the pressure drop for one phase flow is much less than for two phase flow.

This section summarized the analysis which was done in order to select the correct pressure transducer. The instrumentation that was eventually used for the relative permeability tests is discussed in chapter six.

5.3 ESTIMATION OF SATURATION DISTRIBUTION

To design the permeability cell, it was necessary to estimate the saturation distribution which would develop. Relative permeability measurements must be made in a region of uniform saturation. This requires a much longer specimen than that used in many conventional permeameters. This problem has been ignored in much previous research, as will be discussed in chapter six. To ensure that a sufficiently long specimen was used for this test, it was necessary to estimate the fluid saturations. The theory regarding this procedure was presented in chapter four. For a specimen of length L , the distance (z) from the inlet to a plane with saturation S_w is described by equation [4.11]. Using the proper subscripts for gasoline and water, [4.11] becomes:

$$z = L - \int_{S_w}^{1 - S_{r'}} \frac{\frac{dP_c}{dS_w}}{\frac{1}{K} \left(\frac{q_w \mu_w}{k_{rw}} - \frac{q_g \mu_g}{k_{rg}} \right)} dS_w \quad [5.16]$$

The solution of [5.16] requires:

- the capillary pressure as a function of saturation
- the relative permeability functions

- the intrinsic permeability
- specification of the water flux and gasoline flux.

The water flux and gasoline flux are the only parameters which can be specified with any certainty. The capillary pressure saturation relation was not known. The relative permeabilities would not be known until testing was done. The target intrinsic permeability was $1 \times 10^{-16} \text{ m}^2$ to give a hydraulic conductivity of $1 \times 10^{-7} \text{ cm/s}$. This would give the shortest test time. However, the actual value would likely be quite variable.

The uncertainty in so many parameters makes it extremely difficult to obtain even a remotely accurate prediction of the saturation distribution. However, reasonable values were estimated in order to predict the order of magnitude of the outlet distance. It was possible that the outlet distance could be several meters in length due to the large capillary pressures which develop in fine grained soils.

A drainage capillary pressure curve for Preston clay was modified for this analysis. The curve was based on data from Richards (1941). The data were expressed as capillary pressure as a function of gravimetric moisture content (w). The data were converted to capillary pressure as a function of water saturation using:

$$S_w = w \frac{G_s}{e} \quad [5.17]$$

Where S_w = water saturation

e = void ratio

w = gravimetric moisture content

G_s = specific gravity of soil solids.

The ratio G_s/e was obtained by substituting the saturated moisture content into [5.17] and setting S_w equal to unity. The moisture contents were then converted to saturations. A curve was fit to the data as shown in Fig. 5.5. The equation of the interpolation curve is:

$$P_{c \text{ a-w}} = 102.2 \text{ kPa} (1.0 - S_w)^2 \quad [5.18]$$

The air-water capillary pressure curve was modified to be a closer approximation of the gasoline-water capillary pressure. Recall from chapter three that for a capillary tube:

$$P_c = \frac{2 \sigma \cos \phi}{r} \quad [5.19]$$

Thus,

$$P_c \propto \sigma \quad [5.20]$$

$$P_c \propto \cos \phi \quad [5.21]$$

No value could be found for the gasoline-water contact angle on mineral grains.

Interfacial tensions were available. For an air-water interface:

Fig. 5.5 Air-water capillary pressure (data from Richards 1941).

$$\sigma_{a-w} = 72 \text{ mN/m} \quad [5.22]$$

For a gasoline-water interface (Mercer and Cohen 1990):

$$\sigma_{g-w} = 50 \text{ mN/m} \quad [5.23]$$

The capillary pressure function was modified by multiplying [5.18] by 50/72 to obtain:

$$P_{c \text{ } g-w} = 70.94 \text{ kPa } (1.0 - S_w)^2 \quad [5.24]$$

This capillary pressure is shown in Fig. 5.6. It should be noted that neither [5.18] or [5.24] are asymptotically vertical at the residual saturation. The water saturation in the test specimen never decreases below $S_{w\infty}$, and thus the interpolation at low saturations is not important.

To solve [5.16], the derivative of [5.24] was taken:

$$\frac{dP_c}{dS_w} = -141.9 \text{ kPa } (1.0 - S_w) \quad [5.25]$$

The water and gasoline fluxes were both set to one half of the target total flux:

$$q_w = q_g = 5.6 \times 10^{-4} \text{ cm/min} \quad [5.26]$$

A very simple relative permeability curve was selected as shown in Fig. 5.7. This curve had to have the same limiting saturations as the capillary pressure curve. An intrinsic permeability of $1 \times 10^{-16} \text{ m}^2$ was specified.

Fig. 5.6 Gasoline-water capillary pressure.

Fig. 5.7 Relative permeability curve.

The water saturation far from the outlet (S_{wm}) was then solved for using the procedure in the previous section. Equation [5.16] was then numerically integrated to obtain $(z - L)$ for various saturations between S_{wm} and unity. It was impossible to solve [5.16] for z directly since the overall length (L) had yet to be selected. The water saturation as a function of distance from the outlet is shown in Fig. 5.8.

Once the water saturation distribution had been calculated, the pressure distribution was calculated using Darcy's law. For the water phase, Darcy's law gives:

$$\frac{dP_w}{dz} = - \frac{q_w \mu_w}{K k_{rw}} \quad [5.27]$$

At the outlet, a back pressure of 200 kPa was applied. The application of a back pressure is one method of ensuring saturation. By continuity of pressure across the outlet, the following boundary condition applies:

$$\text{At } z = L, \quad P_w = 200 \text{ kPa} \quad [5.28]$$

Integration of [5.27] with the boundary condition from [5.28] gives:

$$\int_{P_z}^{200 \text{ kPa}} dP_w = \int_z^L - \frac{q_w \mu_w}{K k_{rw}} dz \quad [5.29]$$

$$P_w = 200 \text{ kPa} + \frac{q_w \mu_w}{K} \int_z^L \frac{1}{k_{rw}} dz \quad [5.30]$$

Similarly, for the gasoline phase

$$P_g = 200 \text{ kPa} + \frac{q_s \mu_s}{K} \int_z^L \frac{1}{k_{rg}} dz \quad [5.31]$$

Equation [5.30] and [5.31] were then numerically integrated to obtain the water and gasoline pressure distributions. These are shown in Fig. 5.9. The capillary pressure was then calculated using:

$$P_c = P_g - P_w \quad [5.32]$$

The capillary pressure distribution is shown in Fig. 5.10.

There are several conclusions which can be drawn from Fig. 5.8, Fig. 5.9, and Fig. 5.10. By definition, the end effect exit length includes that region where the saturations and the capillary pressure are changing. Based on this analysis, the exit length is approximately 20 mm. The drastic variations in saturation and capillary pressure are confined to a distance of 10 mm from the outlet. The exit length is also apparent in Fig. 5.9. The pressure gradients are constant (ie-linear pressure variation) up to approximately 10 mm from the outlet. The two pressure distributions are also parallel up to this point.

From these observations, the test section must be at least 10 mm from the outlet. Ideally, at least 20 mm would be left between the downgradient pressure transducer and the outlet. As well, when the specimen is extracted for saturation measurement, the 10 mm to 20 mm outlet section should not be used.

Fig. 5.8 Water saturation distribution.

Fig. 5.9 Water and gasoline pressure distributions.

Fig. 5.10 Capillary pressure distribution.

This simulation is not intended to represent an actual estimation of the test system. This simulation involved too many assumptions. The simulation was intended to provide an order of magnitude estimate of the outlet length. The most important conclusion of this work is that under certain conditions, the outlet distance can be confined to a 10 mm to 20 mm distance from the outlet. It is feasible to perform a relative permeability test on a clay soil with a specimen that is on the order of 100 mm long.

5.4 SUMMARY

Analytical modelling of the proposed design was undertaken for two reasons. In order to purchase the correct pressure transducer, it was necessary to estimate the pressure drop to be measured over a 25 mm test length in the region of uniform saturation. Several combinations of material properties were used due to uncertainty in the actual values.

In order to ensure that pressure and saturation measurements were made in the region of uniform saturation, it was necessary to estimate the saturation distribution in the specimen. A great many assumptions were made to obtain this distribution. The results provide an order of magnitude estimate of the outlet distance.

CHAPTER 6

MATERIALS AND METHODS

6.1 INTRODUCTION

This chapter describes the test apparatus which was used to obtain the relative permeability curve of a clayey soil permeated with an organic liquid. The apparatus and the test procedure were modified many times. These modifications will be described when they help to explain the final design. Eventually, a final design and test method were developed. A complete set of drawings of the apparatus is provided in Appendix A.

This permeameter is composed of several components:

- the two phase pump
- the test cell
- the pressure measurement instrumentation
- the saturation determination apparatus.

A pump is required to deliver two liquids at known flow rates. The two fluids flow through soil in a test cell. The pressure drop is measured as the two fluids flow through the test section of the soil in the test cell. The specimen is then extracted. The fluid saturations are measured in the test section of the soil.

The test method includes:

- mixing of soil and compaction/consolidation
- saturation of soil
- relative permeability testing
- extraction of soil and saturation determination.

6.2 PERMEAMETER

6.2.1 General

The objective of this research is to obtain the relative permeability curve for two phase flow through fine grained soils. To accomplish this, the technology developed in two separate fields was combined. In chapter two, the common permeameters used for permeability testing of low permeability soils were described. The main issues for these test methods were elimination of sidewall leakage, and the simulation of the *in situ* condition with regards to the development of shrinkage cracks. In chapter four, the common relative permeability test methods for two phase flow were described. The main issues for these test methods were elimination of inlet and outlet end effects. The permeameter described in this section had to incorporate aspects of both types of tests.

6.2.2 Two phase pump

A basic requirement of relative permeability testing is to pump two phases into the soil specimen. It must also be possible to deliver these phases at different flow rate ratios. For permeability testing of fine grained soils, it must be possible to pump the permeant

at very low flow rates to avoid excessively large gradients. An additional decision that has to be made is whether to use a constant head or a constant flow rate. Most rigid wall (compaction mold) and flexible wall (triaxial) permeameters use a constant head. The inlet fluid is pressurized to a known value. The permeability is calculated using the known gradient and measured flow rate. The permeability testing by Fernandez and Quigley (1985) used a constant flow rate test. Their test used a syringe mounted in a triaxial frame to generate the constant flow rate. It is more convenient to measure the pressure than to measure flow rates. Flow rates are typically measured by collecting the outflow into graduated burettes. However, evaporation of liquid from the burette is a problem if low flow rates are used. Also, since the outflow contains two fluids for relative permeability tests, measurement of both flow rates is more difficult.

The pump used here is a modification of that used by Fernandez and Quigley (1985). The triaxial frame used here can operate at 42 speeds between 0.00005 mm/min and 4.0 mm/min. The triaxial frame has a stroke of 100 mm. This system is designed to provide two phase flow at variable flux ratios. Eight 38 mm (1.5 inch) diameter syringes were mounted in the triaxial frame. The pump can provide flow rates between 4.6×10^{-4} mL/min and 36.5 mL/min. With the stroke of 100 mm, a total of 912 mL of fluid can be pumped before the pump must be recharged. For the soil that was eventually used for this research, the pump was able to push 18.6 pore volumes of flow through the test section. This should be sufficient to reach chemical equilibrium. A detail of one syringe is shown in Fig. 6.1. The syringe has a 38 mm (1 inch) diameter brass plunger.

Originally, steel plungers were used, but they started to rust and risked compromising the integrity of the seals. The syringe cylinder is made of MC 901 nylon. A double o-ring seal is provided at the top of the bore. Below the seals, the cylinder bore diameter is enlarged by 0.64 mm (0.025 inch). This is to allow for swelling of the nylon. Nylon was used because it is relatively inexpensive compared to other materials. Nylon is also strong, can be machined quickly, and can be machined to fairly close tolerances. However, the design details must consider that nylon swells when it absorbs water. Originally, the cylinder bore was uniform. The o-ring seal was placed on the tip of the plunger. The nylon cylinder absorbed water, swelled, the inside diameter was reduced, and the pump started to bind. The solution involved enlarging the cylinder bore so that the inside diameter after swelling was still larger than the plunger diameter. The o-ring seals were then placed on a short section of unenlarged cylinder at the top. The cylinders socket into a 25 mm (1 inch) thick stainless steel plate. All of the materials used in this project had to be selected with consideration to compatibility with water and gasoline. Fluid exits the syringe through a hole in the base plate, and flows into flexible tubing. To keep the syringe cylinders socketed into the pump base plate, the cylinders are sandwiched between the aluminum top clamping plate and the stainless steel base plate. The base plate and top clamping plate are held together by tie rods. A 25 mm (1 inch) thick steel plate takes the reaction from the top of the plungers. The base plate, cylinders, and top plate are all moved upwards by the triaxial frame. The plungers stay in place. The result is a reduction of the volume in the syringes, forcing fluid out. The entire assembly is shown in the triaxial frame in Fig. 6.2. To fit the eight syringes in the

Fig. 6.1 Detail of a syringe.

Fig. 6.2 Syringe pump in triaxial frame.

triaxial frame, it was necessary that they be placed in two rows of four syringes per row.

The outlet of each syringe can be connected to one of two manifolds. This arrangement is shown in Fig. 6.3. All of the syringes that are filled with water are connected to the same manifold. All of the syringes filled with the organic liquid are connected to the other manifold. By varying the number of syringes connected to each manifold (and also the number of syringes filled with each fluid) the ratio of water flow rate to organic flow rate can be varied. Each manifold is designed to accept between zero and eight syringe outflows. The two manifolds are identical, and the design is shown in Fig. 6.4. The manifolds are made of stainless steel. Ports that are not being used during a given test can be plugged. The outflow is through a valve in the face of the octagon. Flexible tubing connects the syringes to the manifolds to allow for movement of the pump base plate in the triaxial frame.

6.2.3 Test cell

The test cell is the most important component of the permeameter since it contains the soil specimen. In chapter two, four types of permeameter test cells were described. The rigid wall (compaction mold), flexible wall (triaxial), and double ring permeameters are quite common. However, the permeameter used by Fernandez and Quigley (1991) is perhaps superior in its simulation of the *in situ* condition. The rigid cell wall with applied overburden was chosen for the relative permeability permeameter used for these tests.

Fig. 6.3 Connection of syringes to manifolds. Arrangement shown here is for three water syringes. The remaining five syringes (one in front row, four in back row) are filled with the organic liquid.

Fig. 6.4 Manifold design.

At this point, it is important to understand why multiphase flow testing in the Fernandez and Quigley permeameter is inappropriate. Fernandez and Quigley (1985) used their permeameter to measure the permeability of a clay soil to benzene using piston type flow of the benzene. Benzene is an LNAPL. Multiphase flow concepts must be used to describe the flow of benzene. However, it could be argued that the tests done by Fernandez and Quigley are conservative. Benzene was forced through the soil as the only flowing phase. Water should have been reduced to its residual saturation. With water at its residual saturation, the benzene effective permeability ($K k_r$) should have been at a maximum and represent the maximum permeability which could exist for the flow of benzene through a landfill liner. There is one problem with this argument. Due to the outlet end effect, the benzene saturation at the outlet must have been at the residual value. The specimen used by Fernandez and Quigley was only 20 mm thick. Recall that in chapter five, the exit length prediction was at least 10 mm, and easily as large as 20 mm. The entire specimen used by Fernandez and Quigley may well have been within the exit length. The entire specimen could have been close to residual benzene saturation, not residual water saturation. This would *not* yield the maximum benzene effective permeability, and would *not* be a conservative test. In addition, the exit length is larger for low ratios of water flux to nonwetting fluid flux (Thomsen 1987). For the tests done by Fernandez and Quigley, the water flux was zero and the exit length should have been at a maximum. Another issue is that Fernandez and Quigley used terminal pressures. Due to the exit effect, these terminal pressures may not have provided reasonable estimates of the actual gradients in the soil. Permeabilities calculated using these

gradients would be inaccurate. In summary, previous tests done to measure the effective permeability for the flow of an organic through clay may not have been conservative, and terminal pressures can not be used.

For these reasons, a longer specimen is used here, and terminal pressures are not used. The test cell is shown in Fig. 6.5. The cell consists of a 101.6 mm (4 inch) inside diameter nylon cylinder sandwiched between two end plates. The inside dimension of 101.6 mm was chosen as that of the Proctor mold, since the sample is compacted using a Proctor hammer. The two fluids flow from the two manifolds in through the base plate. Upward flow is used to facilitate saturation. Downward flow can trap air bubbles. The two end plates are held together by tie rods. The outflow is through the piston head. The entire assembly is placed in an oedometer (consolidation testing apparatus). The oedometer is used to squeeze the soil between the base plate and the piston head. Weights are applied on an 11:1 lever arm to provide substantial compressive stress. Permeability testing of fine grained soils generally requires large gradients to reduce the test time. Large gradients generate large pore fluid pressures. Unless a substantial surcharge is applied, the effective stress can drop to zero and piping can occur. Piping is the process whereby soil particles are carried away with the pore fluid. The surcharge also provides a more realistic approximation of the *in situ* stress condition. If shrinkage cracks form, they will not be immediately closed as happens in the flexible wall permeameter. Shrinkage will also not automatically result in sidewall leakage as happens in the rigid wall permeameter. Unlike the compressed spring used by Fernandez and

Quigley (1991), the oedometer maintains a constant surcharge even when the specimen shrinks.

The soil specimen is separated from the base plate and piston head by 3.2 mm (1/8 inch) thick porous brass discs. The porous brass discs stop soil particles from clogging the inlet and outlet orifices. The soil specimen was approximately 65 mm in height. Pressure measurement was made through two pressure taps into the side of the test cell cylinder. To stop soil from clogging the holes for the pressure taps, 70 μ m porous polyethylene rings separate the soil from the 1.6 mm (1/16 inch) diameter hole to the pressure tap. A small porous plug could have been used instead of a ring around the entire specimen. However, the ring allows measurement of an average pressure around the entire circumference. The use of so-called piezometer rings has been found to give less erratic pressure readings than a pressure measurement at only one point on the circumference (Wyckoff and Botset 1936). The use of piezometer rings necessitated that the test cell cylinder be made in three components that socket together. The sockets have o-ring seals. This system requires a fairly thick wall cylinder to allow room for the piezometer rings and the seals. The most economical material was found to be the same material used for the syringe cylinders, MC 901 nylon. The cylinder must be tightly squeezed to stop fluid pressure from separating the three sections.

A 10 mm inlet length is provided between the top of the inlet porous disc and the first piezometer ring. This distance is provided to encourage fluid mixing and one dimensional

- A - TOP PLATE
- B - OEDOMETER LOADING PIN
- C - ELBOW FITTING FOR OUTLET TUBING
- D - PISTON HEAD (STAINLESS STEEL)
- E - CYLINDER (NYLON)
- F - PRESSURE TAPS
- G - PORT FOR INFLOW FROM MANIFOLD
- H - POROUS BRASS DISCS
- I - POROUS POLYETHYLENE PIEZOMETER RINGS
- J - BASE PLATE
- K - TIE ROD

Fig. 6.5 Test cell.

flow. The piezometer rings are spaced 25 mm apart, providing a test section of this length. An exit length of approximately 25 mm is provided. Recall from chapter five that the exit length based on the analysis assumptions was no more than 20 mm, and close to 10 mm. If possible, a greater exit length would have been provided since the analysis was based on assumptions of unknown accuracy. However, the consolidation apparatus can just barely accept the test cell with a 25 mm exit length. The test cell fits into the oedometer with only 1 or 2 mm to spare.

With such a short entrance length (10 mm), it is crucial that the fluids enter the specimen in an approximately one dimensional flow pattern. This required careful design of the base plate, which is shown in Fig. 6.6. The unusual shape of the base plate is to fit around the hanger rods of the oedometer. The base plate sits on the base plate of the oedometer and thus the fluids must enter the test cell base plate at its edge. The tubing arrangement for the two fluids is shown in Fig. 6.7. The organic liquid (coming from the organic liquid manifold) enters the base plate at port B (see Fig. 6.7). The organic liquid then flows through a tube in the plate, and exits into the test cell on the central axis. In order to distribute the fluid more evenly, a series of grooves are provided in a similar pattern to the dispersed feed apparatus described in chapter four. Water (coming from the water manifold) is split into two tubes at a tee fitting just before entering the base plate. The two streams enter the base plate at ports A and C (see Fig. 6.7). Each stream enters the test cell at two orifices, providing a total of four points of water entry. The four orifices are placed at the centroids of the quarter circles formed by the organic liquid

Fig. 6.6 Test cell base plate.

Fig. 6.7 Tubing arrangement for flow into the test cell base plate.

dispersion grooves. This arrangement attempts to provide even distribution of the water across the cross section. Additional fluid distribution should occur as the fluids flow through the porous brass disc and the 10 mm inlet length.

The cylinder sockets into the base plate in a 3 mm deep groove with an o-ring seal. A similar groove is provided in the top plate. No seal is provided for the top plate since it serves only to hold the cylinder together. Outflow is through the piston head. A double o-ring seal is provided between the piston head and the cylinder. A major concern was that fluid would leak through this interface. Tight tolerances were specified for the cylinder inside diameter. No leakage occurred. Grooves are provided in the piston head face to provide as close to one dimensional flow as possible in the outlet region. The piston head is shown in Fig. 6.8. Fluid flows through a hole in the piston head and through tubing which passes through a hole in the top plate. There is very little room to spare in this arrangement. The elbow fitting on top of the piston head must be within a very limited range of orientations. The placement of the test cell in the oedometer is shown in Fig. 6.9.

Fluid flows out of the test cell with both phases in the same tubing. The fluids then pass into a reservoir. This reservoir is shown in Fig. 6.10. The top of the reservoir is pressurized to provide a back pressure to the system. Application of a back pressure is common in flexible wall (triaxial) permeability testing. The back pressure likely functions in two ways. The increased pressure increases the solubility of the air bubbles in water.

Fig. 6.8 Piston head.

Fig. 6.9 Test cell in oedometer.

Fig. 6.10 Outlet reservoir.

The air bubbles may also compress and take up less void volume. The result is increased fluid saturations. A back pressure of 200 kPa was applied.

6.2.4 Pressure measurement instrumentation

Two pressures are measured during these tests. The pressure measurement instrumentation is shown in Fig. 6.11 in connection with the entire system.

The pressure drop in the test section is measured using a differential pressure transducer. Two differential pressure transducers were obtained. One transducer has a limit of 103 kPa (15 PSID), and the other has a limit of 207 kPa (30 PSID). Based on the analysis from chapter five shown in Fig. 5.4, it was felt that the flow rate could be adjusted so that all tests could be completed in a reasonable time using these pressure transducers. The transducers are card mountable. Both transducers were mounted in an electronics project box. Bulkhead union swagelok fittings provide connection ports. Either transducer can then be connected to the test cell with braided vinyl tubing. The output is measured with a voltmeter and recorded using a chart recorder. The chart recorder allows tests to be run unattended for several days. Calibration curves for both differential pressure transducers are provided in Fig. 6.12 and Fig. 6.13.

A pressure transducer is attached to the outlet from the nonwetting fluid manifold. This transducer measures the pressure of the nonwetting fluid before it enters the test cell. It was originally intended to measure the displacement pressure using this transducer.

- A - DIFFERENTIAL PRESSURE TRANSDUCER AND CHART RECORDER
- B - ORGANIC LIQUID INLET PRESSURE TRANSDUCER

Fig. 6.11 Pressure measurement instrumentation.

Fig. 6.12 Calibration curve for 30 PSID differential pressure transducer.

Fig. 6.13 Calibration curve for 15 PSID differential pressure transducer.

Recall from chapter three that the displacement pressure is the capillary pressure required to initiate entry of a nonwetting fluid into a pore. It was intended to measure the peak nonwetting fluid pressure required to initiate flow. However, it was later realized that the displacement pressure could not be measured using this transducer. There are two reasons why the displacement pressure can not be measured. First, it is necessary to know the pressure of the water being displaced since the displacement pressure is a pressure difference between the two fluids. No instrumentation was available to measure this water pressure. The second problem is that there was no pressure spike or jump that could be attributed to the initiation of flow into the soil. There was a gradual ramping upward of pressure up to a steady value. As a nonwetting fluid tendril pushes its way into the soil, the measured inlet pressure results from the displacement pressure at the tip of the tendril and from the pressure gradient along the tendril flow path. As the tendril reaches the end of the specimen, the inlet pressure is equal to the back pressure (200 kPa) plus the integration of the pressure gradient along the flow path. This type of constant flow rate test can not be used to measure the displacement pressure since flow is forced. To measure the displacement pressure, the nonwetting fluid must be pressurized just until flow is initiated. The calibration curve for this transducer is shown in Fig. 6.14.

6.2.5 Saturation determination apparatus

In order to develop a relative permeability curve, every relative permeability measurement must be associated with a pair of saturations (S_w , S_g). Recall that:

Fig. 6.14 Calibration curve for 200 PSI inlet pressure transducer.

$$S_g = 1 - S_w \quad [6.1]$$

Also recall that the ratio of the saturations is not equal to the flux ratio:

$$\frac{S_w}{S_g} \neq \frac{q_w}{q_g} \quad [6.2]$$

In other words, the fraction of voids occupied by a fluid is not equal to the fraction of the flowing volume composed of that phase. Three different methods were considered for measuring the fluid saturations. All three methods involve extraction of the specimen following the test, and removal of a portion of the specimen for analysis.

The first method involved distillation of the pore fluids from the soil. The apparatus is basically that described by ASTM E 123 which describes the apparatus for the determination of water by distillation. The apparatus is shown in Fig. 6.15. The soil specimen is placed in an aluminum tar still. This type of still was advantageous for the distillation of gasoline since it can contain an explosion. The still is then heated in a paraffin oil bath to a temperature sufficient to volatilize all liquids. The vapours rise up a condenser. The vapours condense and drop down into a trap. The trap is a round bottom flask. All connections are 24/40 ground glass. During initial trials, vapours were condensing in the tee and dropping back into the still. The tee was insulated with foam water pipe insulation. An insulated cover was made from 2 inch styrofoam house insulation. This cover encased the top of the oil bath, the tar still, and the tee. A plywood spacer had to be placed between the oil bath and the styrofoam insulation to stop the insulation from melting. The insulation succeeded in providing 99.9% water

Fig. 6.15 Distillation apparatus.

extraction and at least 85% gasoline extraction. However, due to problems with the test that will be described later, gasoline was not used for the permeability testing. Vegetable (canola) oil was used. This liquid is not volatile. The oil burns before it can evaporate. Distillation could not be used for saturation determination.

The second technique which was considered was as follows. The specimen was heated to a sufficient temperature to evaporate the water but not burn the oil. The remaining soil with vegetable oil was centrifuged with a solvent (varsol). The difference between the oven dry weight of the soil before and after centrifuging should be the weight of oil in the soil. However, due to loss of soil during centrifuging, this method was not considered accurate.

The third method was gravimetric. This technique was used for the majority of the testing. This method will be described in the description of the test procedure that follows.

6.3 TEST PROCEDURE

6.3.1 General

This section summarizes the test procedure which was developed. The procedure involves four steps:

- 1) Mixing of soil and compaction/consolidation in test cell
- 2) Saturation of soil

- 3) Relative permeability testing
- 4) Extraction and saturation determination.

6.3.2 Mixing of soil and compaction/consolidation

The soil components were mixed dry. For all of the tests, a mixture of No. 70 silica sand and kaolin clay were used. The silica sand and clay were mixed in a bowl to specified proportions. Before the soil could be compacted, the test cell had to be assembled.

The porous plastic rings and porous brass discs were vacuum saturated. It was necessary to ensure that the braided vinyl tubing that connects the pressure transducer to the pressure taps was also saturated. The bottom section of the test cylinder was socketed into the base plate (with o-ring seal in place). The transducer end of the braided vinyl tubing was connected directly to the pressurized outlet reservoir. Water was forced into the tubing until water flowed into the test cell cylinder, as shown in Fig. 6.16. The saturated porous ring and o-ring seal were then put in place and the next cylinder section installed. Water was then forced again through the same tubing to ensure saturation. The tubing was disconnected from the reservoir and plugged. This procedure was repeated for the other pressure tap tubing. Inlet ports A, B, and C on the test cell base plate were also saturated using flow from the reservoir. The bottom porous brass disc was installed. The cylinder was then sandwiched between the bottom and top plates without installing the piston head. This left an 80 mm diameter hole in the top plate through which soil could be added.

Fig. 6.16 Saturation of pressure transducer tubing.

The dry soil was then moistened with a specified amount of water. A sample was taken to verify the compaction moisture content. The soil was placed in the test cell in three layers with each layer being compacted with 25 blows per layer of the standard Proctor hammer.

Originally, it was attempted to trim the specimen, install the piston head and run a test. However, when the overburden load was applied with the oedometer, the sample consolidated more. The consolidation was often to such an extent that the oedometer weights were resting on the table with no more room to travel. To rectify this problem, all samples were consolidated after being compacted. Consolidation involved removing the top plate, installing a thin loading plate on the soil, and replacing the top plate. The loading plate is a thin stainless steel plate with a diameter slightly less than the test cell inside diameter. This assembly is shown in Fig. 6.17. The test cell was placed in the oedometer. A load of 47.7 kg was then placed on the oedometer, giving a stress of 635 kPa in the soil. The sample was allowed to consolidate for 24 hours under this load, which was the same overburden as applied during the test. Port B was opened during consolidation to allow drainage. After 24 hours, the test cell was removed from the oedometer. The top plate and loading plate were removed for the test cell. The soil was then trimmed to the required length by scraping away excess soil with a spatula. The top porous brass disc was installed. The 25 mm (1 inch) thick piston head with greased double o-ring seals was installed. The top plate was reinstalled. The entire assembly was placed in the oedometer. Since the soil had been consolidated for 24 hours under the

Fig. 6.17 Consolidation of soil sample.

maximum stress to be applied during the test, very little compression occurred as the surcharge was reapplied. This was important so that the weights would not hit the table. Also, since the test cell just barely fits into the oedometer, excessive compression could result in the top plate coming into contact with the oedometer. The outlet tubing was connected, and the pressure tap tubing connected to the appropriate transducer.

The procedure which was just described is applicable to a soil which can be compacted with a Proctor hammer. The procedure was modified for soils with a high clay content. Such soils were placed in the test cell in three layers, with each layer being rodded 25 times with a spatula to remove large air voids. The loading plate was installed and the test cell placed in the oedometer. If the entire surcharge was then applied to consolidate the clay, clay would ooze out through the gap between the loading plate and the cylinder wall. Load had to be applied in steps. With each load increment, the soil would consolidate and gain strength. Eventually, the clay could be consolidated under the entire load.

This procedure of mixing, then compaction, then consolidation produced excellent contact between the soil and the test cell wall. At one point, it was attempted to compact and consolidate the soil specimens in a Proctor mold. The specimen could then be transferred to the test cell. This would have allowed a specimen to consolidate while another was being tested. However, side wall leakage was a problem. There was obviously poor contact between the soil and the test cell wall. This was recognized by a measurement

of zero differential pressure for flow through an obviously low permeability soil. Water was flowing around the specimen. As a result, all specimens were compacted and consolidated in the test cell.

6.3.3 Saturation of soil

Saturation was achieved by pumping several pore volumes of water through the soil. A back pressure of 200 kPa was applied to help in saturating the sample.

The first step in the saturation procedure was to fill the pump. A major concern was that air bubbles would be trapped when the pump was filled and that these air bubbles would be pushed into the soil. To avoid trapping air bubbles, the pump was filled in the following manner. The syringe plungers were withdrawn so that the syringes were open to the atmosphere. The plungers were removed by lifting up the plungers and top plate assembly with the large locking nuts which secure the triaxial frame reaction beam to the vertical screws. This is shown in Fig. 6.18. The outlet reservoir was filled with water and connected to the water manifold. The outlet reservoir was pressurized, forcing water to flow into the pump in the opposite direction to which water flows during a test. All eight syringes could be filled with water this way, or fewer if desired. If some of the syringes had been oil filled in the previous test and would be oil filled for this test, these syringes could be left empty during the saturation stage. A minimum of four syringes were used for saturation to ensure that a sufficient volume of water was forced through the soil. The manifold valve was closed, and the plungers reinstalled. The outlet

Fig. 6.18 Filling water syringes using reservoir.

reservoir was depressurized and the manifold valve opened. The weight of the plungers, the top plate, and the triaxial frame reaction beam then pushed some water out of the syringes and back into the reservoir. The plungers had to move far enough into the syringes that the o-rings could seal against the plungers. The manifold valve was closed and the manifold was disconnected from the reservoir.

The manifold was connected to all three inlet ports on the test cell base plate using a tee fitting and some additional tubing. The test cell outlet was connected to the outlet reservoir. The valves were opened and water pumped into the soil specimen. At first, only air was displaced from the soil. Eventually, only water would come out of the specimen. All mobile air had been removed. At this point, the back pressure was applied in gradual steps to avoid surging flow. Surges in flow were found to result in piping.

Generally, a steady state pressure drop was recorded quite early during saturation. The triaxial frame loading head speed was recorded. The nominal loading head speeds were not found to be accurate. The distance between the aluminum clamping plate and the top plate was measured with callipers several times during the test. The speed was calculated from the change in separation with time. The intrinsic permeability was calculated using the flow rate and pressure drop. Saturation continued until the pump reached the end of its stroke.

6.3.4 Relative permeability testing

Relative permeability testing was accomplished by pumping both water and oil through the soil in a fixed ratio. For example, if the water to oil flux ratio was to be 3:5, three syringes were filled with water, and five with oil.

To prepare for the relative permeability test, the pump had to be retracted. The reservoir had to be emptied and filled with fresh water to be used to fill the pump. This required that the test cell be detached from the outlet reservoir and from the water manifold. Recall that at the end of saturation, the test cell was at 200 kPa from the back pressure. It was found that if the test cell was depressurized between subsequent saturation stages, the permeability changed. To avoid this change in intrinsic permeability, valves were installed so that the test cell need not be depressurized when detached from the reservoir or manifolds. This is shown in Fig. 6.19.

The pump was then retracted and the specified number of water syringes attached to the water manifold. The plungers were withdrawn from the syringes as before. The reservoir was then used to fill the water syringes as was done for the saturation stage. The water manifold valve was then closed, and the reservoir detached. The reservoir was depressurized, drained, and filled with oil. The reservoir was then attached to the oil manifold and used to fill the oil syringes in the same manner as for the water syringes. The oil manifold valve was closed, and the reservoir depressurized. The plungers were dropped back into place and the manifold valves opened. The weight of the plungers and

Fig. 6.19 Procedure for filling pump without depressurizing test cell.

top plate pushed fluid out of the syringes. Water was forced out the water manifold outlet and was collected. Oil was forced into the now depressurized reservoir. This procedure resulted in no air bubbles being trapped in the tubing.

Originally, the pump was reattached to the test cell at this point. However, as the valves were opened, the pressure in the test cell forced fluid into the pump. There were obviously air pockets just below the plungers. These air pockets compressed when pressurized. The sudden surge of flow back into the pump was sufficient to dislodge soil and result in piping. In addition, the test cell lost pressure. The solution was to pressurize the pump after the plungers had been dropped into place. The reservoir was used to pressurize both the water and oil in the pump to 200 kPa. The manifold valves were then closed and the manifolds connected to the appropriate test cell inlet ports. Before any valve was opened, the reservoir, test cell, and pump were all at 200 kPa. The valves were opened and the pump turned on. The pressure difference and pump speed were recorded as during saturation.

When the test was complete, there were two options as to what followed. The first option was to extract the specimen and determine the fluid saturations. This process is described in the following section. The problem with extracting the specimen is that the relative permeability at other flux ratios would be determined on different soil specimens. The second option was to seal off the specimen, fill the syringes at a different ratio, and run another test. The advantage of this option is that all of the relative permeabilities are

measured with the same soil specimen. The disadvantage is that the saturations associated with each relative permeability are not determined.

6.3.5 Specimen extraction and saturation determination

The following section describes the gravimetric method for determining the fluid saturations. This method was found to be the most reliable for oil and water saturated specimens. Following the relative permeability test, the back pressure was reduced to zero and all tubing detached from the test cell. The overburden was removed and the test cell removed from the oedometer. The top plate and base plate were removed from the test cell cylinder. To extract the soil specimen, it was necessary to use a hydraulic jack to push the soil and piston head out the top of the cylinder. The porous discs were removed from the soil specimen and the soil placed into a Proctor mold. A Shelby tube sample was taken from the centre of the soil specimen. The full height of the soil specimen was sampled. A sample was taken of the annular ring of soil remaining in the Proctor mold. This sample was placed in an oven dry and previously weighed evaporation dish. The soil in the Shelby tube was extruded using a hydraulic jack. The first 15 mm (approximately) of soil extruded from the Shelby tube were from the outlet end of the specimen. This part of the sample was put in a previously weighed evaporation dish. The remaining length of the sample was also placed in a weighed evaporation dish. All three evaporation dishes were weighed and placed in an oven at 110 °C. The dishes were removed from the oven 24 hours later and weighed again. The method of determining the saturations from this data is presented in chapter seven.

CHAPTER 7

RESULTS AND DISCUSSION

7.1 INTRODUCTION

In this chapter, the results of the laboratory testing are presented in generally chronological order. Ancillary tests which were undertaken to evaluate the effect of flow rate and back pressure are presented. A substantial amount of trial tests were performed during the lengthy de-bugging stage of this research. The test method presented in chapter six was arrived at after many weeks of adjustments. These trial tests were presented in chapter six when they helped to explain the development of the test procedure. These tests are not reiterated here.

As mentioned previously, an unfortunate characteristic of permeability tests on fine grained soils is that they are generally of long duration. This severely limits the amount of data which can be obtained.

All of the relative permeability tests carried out here are drainage tests. The specimens were water saturated, then tested. If several flux ratios were used on the same specimen, the high water to oil flux ratios were used first. This way, the water saturation was always decreasing. Thus, all data points are for the drainage relative permeability curve. The drainage relative permeability curve is the more important curve for contaminant transport applications since it applies to the condition of an organic chemical permeating

into a soil, and displacing water in the process.

7.2 SELECTION OF SOIL MIXTURE AND PERMEANTS

The design of the permeameter was based on the intended use of gasoline and water as permeants and the use of a soil with a hydraulic conductivity of 1×10^{-7} cm/s. During the test procedure development stage, a higher permeability soil was used ($k = 10^{-6}$ cm/s). As part of this process, a trial relative permeability test was run with gasoline and water. Three problems were encountered.

The differential pressure measurement was erratic. A schematic of the differential pressure as a function of time is shown in Fig. 7.1. The flow through the test cell was obviously unsteady, even after several pore volumes of flow. A hypothesis to explain these measurements is as follows. Gasoline is a low viscosity liquid and is less dense than water. When the gasoline entered the test cell, it might have immediately risen up through the porous brass disc as bubbles. The gasoline might not have been filling the dispersing grooves, and might not have been flowing into the soil as a continuous phase. As these bubbles pass a pressure transducer, they would likely generate transient pressure spikes. The differential pressure transducer would register an increase in differential pressure as the bubbles passed the upgradient pressure tap, and then register a decrease as the bubbles passed the downgradient pressure tap. This hypothesis is shown in Fig. 7.2. To overcome this problem, several possible actions were considered. A higher flow rate might have prevented bubble formation, but the test might not reach steady state in

Fig. 7.1 Erratic transducer output during gasoline test.

Fig. 7.2 Hypothesis regarding erratic pressure measurements.

3 of/de 3

PM-1 3½"x4" PHOTOGRAPHIC MICROCOPY TARGET
NBS 1010a ANSI/ISO #2 EQUIVALENT

the short time before the pump reached the end of its stroke. The other possibilities were to use a higher viscosity organic liquid, or a lower permeability soil. Both of these options would probably prevent bubble formation.

The option of using a higher viscosity organic liquid was selected due to two other problems encountered with the use of gasoline. This test system is well designed for use with volatile noxious liquids since the liquids are completely sealed in the pump, test cell, and reservoir. However, during filling of the pump, the gasoline fumes were a severe irritant. As well, the tubing between the test cell and the reservoir split during one overnight test. The back pressure from the outlet reservoir sprayed gasoline into the lab. The bubble formation, noxious fumes, and tubing failure all contributed to the decision to select a different organic liquid. Vegetable oil (canola) was selected due to its high viscosity, inexpensiveness, and lack of noxious qualities. Due to the higher viscosity, specimens with higher permeabilities had to be used. Large gradients or long test times would have been necessary if vegetable oil was permeated through a soil with a hydraulic conductivity of 1×10^{-7} c n/s.

The dynamic viscosity of the vegetable oil was measured in accordance with ASTM D446. The measured viscosity was:

$$\mu_o = 5.80 \times 10^{-2} \text{ Ns/m}^2 \quad [7.1]$$

The unit weight of the oil was determined by measuring the weight gain of a 500 mL volumetric flask when filled with the oil. The measured unit weight was:

$$\gamma_s = 8.96 \text{ kN/m}^3 \quad [7.2]$$

The soil mixture was varied several times. However, the mixture was always composed of No. 70 silica sand and kaolin clay. The grain size distribution of the silica sand is shown in Fig. 7.3.

7.3 ANCILLARY TESTS

The two main test parameters which were controllable were the flow rate and the back pressure. The main concern with the flow rate was that the test be carried out within the range of laminar flow. A test was carried out in which the water flow rate was varied and the pressure drop was measured. A plot of the flow rate as a function of hydraulic gradient is shown in Fig. 7.4. The linear relationship between flow rate and hydraulic gradient would seem to confirm that the flow is laminar within this range of flow rates.

In a separate test, the back pressure was varied and the pressure drop recorded. The results are shown in Fig. 7.5. The permeability increases with increasing back pressure. These results agree with the theory that increasing back pressure increases saturation and increases the permeability. Above 200 kPa, increasing back pressure has very little effect and the soil can be considered to be saturated.

Fig. 7.3 Grain size distribution of No. 70 silica sand.

Fig. 7.4 Water flow rate as a function of hydraulic gradient.

Fig. 7.5 Intrinsic permeability as a function of back pressure.

7.4 FIRST RELATIVE PERMEABILITY TEST

The first relative permeability test on a sample which had been compacted and consolidated as described in chapter six, was conducted on a soil with 15% clay, 85% No. 70 silica sand, and a compaction moisture content (w_c) of 7.0%. Several different soil mixtures had been permeated with water before this mix was obtained with an intrinsic permeability of $1.7 \times 10^{-15} \text{ m}^2$. This corresponds to a hydraulic conductivity of $1.7 \times 10^{-6} \text{ cm/s}$, roughly one order of magnitude greater than the originally intended value of 10^{-7} cm/s .

This test was run without extracting the soil specimen after each relative permeability test. The relative permeabilities at the different flux ratios ($q_w:q_o$) were determined on the same soil specimen. For each flux ratio, the total flow rate was the same:

$$Q_t = Q_w + Q_o = 2.5 \times 10^{-8} \text{ m}^3/\text{s} \quad [7.3]$$

Where Q_t = total flow rate

Q_w = water flow rate

Q_o = oil flow rate.

Although this test did not provide the corresponding fluid saturations, it did provide the overall shape of the relative permeability curves. The use of a constant total flow rate was convenient since each relative permeability test took the same amount of time. Each test took approximately 10 hours. Two tests were run per day. With 24 hours required for consolidation, and nine flux ratios to be used, six days were required to obtain one curve.

The measured steady state pressure drop for the different flux ratios is shown in Fig. 7.6. This pressure drop is conventionally applied to both fluids as discussed in chapter four. For each flux ratio, the gradients in the water and in the oil were calculated using:

$$\frac{dh_w}{dz} = \frac{-1}{\gamma_w} \frac{\Delta P}{\Delta z} + 1 \quad [7.4]$$

$$\frac{dh_o}{dz} = \frac{-1}{\gamma_o} \frac{\Delta P}{\Delta z} + 1 \quad [7.5]$$

Where h_w, h_o = total head in the water and oil respectively

γ_w, γ_o = unit weights of water and oil

ΔP = measured steady state pressure drop

Δz = distance over which pressure drop was measured

= 0.025 m

The unit weight of water was taken as being 9.81 kN/m³. The elevation head gradient of unity was generally negligible in comparison to the pressure head gradient. The gradients in the two fluids are shown in Fig. 7.7 for the various flux ratios. The test for a flux ratio of 8:0 was the water saturation test. The intrinsic permeability was calculated from the saturation test using:

Fig. 7.6 Measured steady state pressure drops at several flux ratios for a soil with $CF = 15\%$, $w_c = 7.0\%$, $K = 1.7 \times 10^{-15} \text{ m}^2$, and a flow rate of $Q_t = 2.5 \times 10^{-8} \text{ m}^3/\text{s}$.

Fig. 7.7 Gradients in water and oil as functions of the flux ratio.

$$K = \frac{-Q_w \mu_w}{\gamma_w \frac{dh_w}{dz} A} \quad [7.6]$$

Where Q_w = flow rate during water saturation

μ_w = water viscosity

= 9.815×10^{-4} Ns/m² (Merritt 1976)

A = test cell cross sectional area

= 8.11×10^{-3} m²

The relative permeability of each fluid at each flux ratio was calculated using:

$$k_{rw} = \frac{-Q_w \mu_w}{K \gamma_w \frac{dh_w}{dz} A} \quad [7.7]$$

$$k_{ro} = \frac{-Q_o \mu_o}{K \gamma_o \frac{dh_o}{dz} A} \quad [7.8]$$

Where k_{rw} , k_{ro} = water and oil relative permeabilities respectively

These calculated relative permeabilities are shown in Fig. 7.8 respectively as functions of the flux ratio. The most interesting observation is that the oil relative permeabilities are all greater than unity. Based on conventional multiphase flow theory, the relative permeability can never exceed unity. One explanation for this phenomenon is that the oil, with its low dielectric constant, could be reacting with the clay portion of the soil. The

Fig. 7.8 Relative permeabilities as functions of the flux ratio.

result of this reaction would be an increase in the intrinsic permeability for the reasons described in chapter two. Permeation with low dielectric constant organic liquids can result in changes in clay soil structure. The changes in soil structure result in increases in the intrinsic permeability. The actual dielectric constant of the oil was not measured since the equipment was not available. However, the dielectric constant is probably lower than that of water. In Fig. 7.8, the water relative permeabilities are too small to see. The relative permeabilities are plotted on different axes in Fig. 7.9. These curves look very much like conventional relative permeability curves, except for the extremely high oil relative permeability values.

An early conclusion that might be drawn from these initial results is that conventional multiphase flow theory is insufficient to describe the flow of an organic liquid through a clayey soil. Darcy's law for multiphase flow is:

$$Q_o = \frac{K \gamma_o}{\mu_o} k_{ro} \frac{dh_o}{dz} \quad [7.9]$$

The only material property which is supposed to be a function of the fluid saturation is the relative permeability. However, it would appear that both the relative permeability and the intrinsic permeability could be functions of fluid saturation.

To determine if this hypothesis was correct, more tests had to be done. Also, it was necessary to determine saturations in order that the true relative permeability curves could

Fig. 7.9 Relative permeabilities as functions of the flux ratio, with relative permeabilities shown on different axes.

be developed. Each flux ratio corresponds to a certain pair of fluid saturations.

7.5 RELATIVE PERMEABILITY TESTS WITH SATURATION DETERMINATION.

To confirm the theory presented in the previous section, more testing was undertaken to confirm the high oil relative permeabilities. Saturation measurements were made so that the true relative permeability curves could be developed.

The same soil mix as for the previous test was compacted and consolidated. However, the hydraulic conductivity was too high (3×10^{-4} cm/s). Variations in the hydraulic conductivity were a problem throughout this testing program. A high hydraulic conductivity required a high flow rate so that the pressure differential would be within the range of the pressure transducer. High flow rates result in short test times and uncertainty as to whether steady state has ever been reached. To avoid this problem, many test specimens had to be rejected. Many days were lost due to this problem since each specimen was consolidated for 24 hours before it could be tested.

After several soil mix modifications, a mix was selected which gave fairly consistent results. This soil mix was used for all subsequent testing in this research. The mix was:

- clay fraction (CF) = 20%
- No. 70 silica sand = 80%
- compaction moisture content (w_c) = 10.3%

Several specimens with this mix were compacted, consolidated, and permeated with water to examine the variation in the permeability. One conclusion was that the intrinsic permeability decreased with increasing consolidation time. This trend is shown in Fig. 7.10. At this point, the consolidation time for all future tests was standardized to roughly 24 hours.

The relative permeability testing which followed involved measuring the relative permeability at different flux ratios on separate soil specimens. In order that the results would be meaningful, the soil specimens would have to be sufficiently similar in terms of soil structure. This is why the grain size distribution, compaction moisture content, and consolidation time were all standardized.

The testing involved compacting a specimen, consolidating for 24 hours, saturating for 10 hours, and performing a relative permeability test for 10 hours. The procedure followed was that described in chapter six. The 10 hours was the time required for the pump to reach the end of its stroke. All tests reached steady state within this time. Thus, each data point took two days to obtain. Eleven data points were collected, requiring approximately three weeks of testing.

Three tests were run with seven water syringes. These three tests took so long (1 week) that all other flux ratios were only tested once. The final test was for residual oil saturation. This state was obtained by saturating the specimen with water, pumping only

Fig. 7.10 Intrinsic permeability as a function of consolidation time for $CF = 20\%$, $w_c = 10.3\%$.

oil to obtain residual water saturation, and then pumping only water to obtain residual oil saturation. These are the only data that were collected for imbibition conditions. All other tests were drainage tests.

The intrinsic permeabilities were calculated for each specimen during saturation. The intrinsic permeabilities were calculated using [7.6]. The compaction moisture contents, intrinsic permeabilities, and relative permeability test gradients are shown in Table 7.1. The relative permeabilities were calculated using [7.7] and [7.8], and are shown in Fig. 7.11 as functions of the number of water syringes. The number of oil syringes is equal to eight minus the number of water syringes. There is no obvious trend to the data in Fig. 7.11. The oil relative permeabilities are, however, all greater than unity, confirming that some reaction may be occurring which increases the intrinsic permeability. Unfortunately, the relative permeabilities for the various soil specimens do not fall on a single curve.

The water relative permeability at residual oil saturation was found to be 0.9. This value, being so close to unity, shows that there has not been a large permanent increase in the intrinsic permeability as might be expected if piping was the cause of the large oil relative permeabilities. A value less than unity is to expected since the residual oil interferes with the flow of water.

Table 7.1 Test data.

No. OF WATER SYRINGES	w _c (%)	K (m ²)	WATER GRADIENT	OIL GRADIENT
7	10.3	6.00x10 ⁻¹⁶	934	1022
7	10.2	9.78x10 ⁻¹⁶	976	1069
7	10.5	7.78x10 ⁻¹⁵	289	317
6	10.3	1.32x10 ⁻¹⁵	296	324
5	10.3	2.15x10 ⁻¹⁵	214	234
4	10.3	9.64x10 ⁻¹⁵	325	356
3	10.4	4.52x10 ⁻¹⁵	688	754
2	10.7	4.33x10 ⁻¹⁶	917	1004
1	10.4	1.01x10 ⁻¹⁴	658	721
0	10.2	8.26x10 ⁻¹⁵	623	682
S _{or}	10.4	1.36x10 ⁻¹⁵	254	N.A.

Fig. 7.11 Relative permeabilities as functions of the number of water syringes.

The saturations were calculated for all of the soil specimens. As mentioned in chapter six, the saturations were measured for three portions of the specimens using a gravimetric technique. The saturations were determined for:

- the annular ring of soil remaining after Shelby tube sampling
- the outlet end 15 mm of the Shelby tube sample (roughly 1/4 of sample length)
- the remaining portion of the Shelby tube sample (roughly 3/4 of sample length)

The upgradient 3/4 of the Shelby tube sample was taken as having the true fluid saturations. The outlet end was sampled to study the outlet end effect. The annular ring was sampled to see if the saturations were uniform across the cross section.

The method of determining the fluid saturations described in chapter six involved measuring the water loss from an oil and water saturated sample when heated in an oven at 110 °C for 24 hours. However, the method of calculating the saturations was not described. The calculation involves the assumption that soil samples with the same grain size distribution, compacted in the same manner, and consolidated for the same length of time should have the same void ratio. The fluid saturations can then be calculated using the measured water content and the known void ratio.

The moisture content was measured for the water saturated specimens in Fig. 7.10 which had been consolidated for 24 hours. The average saturated moisture content was 11.7%. The void ratio was then calculated using (Das 1990):

$$S e = w G_s \quad [7.10]$$

Where S = water saturation
 e = void ratio
 w = gravimetric moisture content
 G_s = specific gravity of soil solids.

For these tests, the saturation (S) at the end of the test was unity. A weighted average value of G_s was calculated using typical values of 2.65 for silica, and 2.6 for kaolin (Das 1990).

$$G_s = 0.8(2.65) + 0.2(2.6) = 2.64 \quad [7.11]$$

The average void ratio was then calculated from [7.10] to be

$$e = (11.7\%) (2.64) = 0.31 \quad [7.12]$$

Since the specimen preparation was identical for the relative permeability tests, the void ratios should be similar.

The fluid saturations were determined using the total mass of the soil samples, the void ratio, and the mass of the soil samples with all water driven off by drying to constant mass (24 hours) in an oven at 110 °C. The mass of the soil after drying in the oven was equal to the mass of the soil solids plus the mass of the oil. Oil was not evaporated or combusted at 110 °C. In the following derivation, the following symbols are used:

M_t = total sample mass

M_w = mass of water in sample

= loss of mass upon drying in oven

M_o = mass of oil

M_s = mass of soil solids

V_w = water volume

V_v = void volume

V_o = oil volume

V_s = volume of soil solids

ρ_w = water density

ρ_o = oil density

ρ_s = density of soil solids

The water saturation following a relative permeability test is

$$S_w = \frac{V_w}{V_v} \quad [7.13]$$

But,

$$V_v = V_w + V_o = \frac{M_w}{\rho_w} + \frac{M_o}{\rho_o} \quad [7.14]$$

Furthermore,

$$M_o = M_t - M_w - M_s \quad [7.15]$$

and

$$M_s = V_s \rho_s \quad [7.16]$$

Recall that

$$G_s = \frac{\rho_r}{\rho_w} \quad [7.17]$$

Substitution of [7.17] into [7.16] gives

$$M_s = V_s G_s \rho_w \quad [7.18]$$

But, by the definition of void ratio

$$e = \frac{V_v}{V_s} \quad [7.19]$$

Substitution of [7.19] into [7.18] gives

$$M_s = \frac{V_v}{e} G_s \rho_w \quad [7.20]$$

Substitution of [7.20] into [7.15] gives

$$M_o = M_t - M_w - \frac{V_v}{e} \rho_w G_s \quad [7.21]$$

The oil mass is now known from measurable quantities. This is important since it can not be measured. Substitution of [7.21] into [7.14] gives:

$$V_v = \frac{M_w}{\rho_w} + \frac{1}{\rho_o} [M_t - M_w - \frac{V_v}{e} \rho_w G_s] \quad [7.22]$$

The mass of oil has been removed from the expression. In [7.22], the void volume is on both sides of the equation. Rearranging, and isolating V_v gives:

$$V_v = \frac{\frac{M_w}{\rho_w} + \frac{1}{\rho_o} [M_t - M_w]}{1 + \frac{\rho_w}{\rho_o} \frac{G_s}{e}} \quad [7.23]$$

The void volume for a sample can now be calculated since all of the terms on the right hand side of [7.23] can be measured or are known. The densities (ρ_w , ρ_o) are known. The water mass (M_w) is the loss of mass upon drying in the oven. The term $M_t - M_w$ is the mass of oil and solids left after the loss of water. The void ratio and G_s have been calculated and are presented in [7.12] and [7.11]. Substitution of the known values into [7.23] gives:

$$V_v = \frac{\frac{M_w}{1 \text{ g/cm}^3} + \frac{[M_t - M_w]}{0.913 \text{ g/cm}^3}}{10.33} \quad [7.24]$$

The water volume can be calculated from

$$V_w = \frac{M_w}{1 \text{ g/cm}^3} \quad [7.25]$$

The water saturation can then be calculated by:

$$S_w = \frac{V_w}{V_v} \quad [7.26]$$

The oil saturation can be calculated by:

$$S_o = 1 - S_w \quad [7.27]$$

This convoluted method for determining the fluid saturations results from the inability to

measure the oil volume directly.

Using this method, the saturations were determined for the individual relative permeability tests. The relative permeabilities as functions of the water saturation are shown in Fig. 7.12. Again, there is no real trend to the data. However, it is interesting to note that the data points are all at high water saturations. This is typical of relative permeability measurements on soils with small pores (Morgan and Gordon 1970). For soils with small pores, two phase flow occurs over a very narrow range of saturations, and all at high wetting phase saturations. In other words, the residual water saturation is quite high, and the saturation difference between residual water and residual oil is small. The high residual water saturation results from the large soil solids surface area in the soil. The soil solids surfaces and the intergrain contact points, are coated with water. The high residual water saturation leaves little room for two phase flow (Morgan and Gordon 1970).

This test was more successful at examining fluid saturations than relative permeabilities. There is too much scatter in the relative permeability data. The ideal results of relative permeability measurements on the same soil specimen (as in section 7.4) combined with saturation measurement requires modification of this equipment. However, the saturations should be fairly consistent for tests with the same flux ratio. It was decided to combine these saturations with the relative permeability measurements on a single soil specimen, similar to the test in section 7.4. This should give the ideal result of a smooth relative

Fig. 7.12 Relative permeabilities as functions of water saturation.

permeability curve, similar to that shown in Fig. 7.9, coupled with saturation measurement. A combination of the best aspects of both types of tests should be achieved. For this reason, it was undertaken to consolidate one more specimen with this mix, and run the full series of flux ratios through it. The results of this test are presented in the next section.

Before discussing the next test, it is informative to consider the saturation measurements in greater detail. Recall that the soil specimen was sampled in three locations:

- upgradient 3/4 of central core
- downgradient 1/4 (approximately 15 mm) of central core.
- annular ring close to the wall of the test cell.

The saturations are shown in Fig. 7.13. Several conclusions can be drawn from these saturations. Although there is a lot of scatter, the upgradient 3/4 core always has the lowest water saturation. The downgradient 1/4 core, corresponding to the outlet section, always has a higher water saturation. This trend can likely be attributed to the outlet end effect discussed previously. Theoretically, the oil saturation is reduced to its residual value at the outlet face, resulting in high water saturations.

The water saturations near the wall are the highest of the three. This means that there is not uniform flow across the cross section. The oil probably rises up from the central orifice without filling the dispersing grooves completely. The base plate design is not adequate in providing one dimensional flow in the test cell. In a related phenomenon, an

Fig. 7.13 Water saturation as a function of the number of water syringes.

important observation was made when removing the cores from the oven after drying. All of the water had been driven off, making the oil much more visible. Since the water saturations were all quite high, there was always very little oil in the core. The oil that was present occurred as blotches. A drawing of the pattern is shown in Fig. 7.14. So, not only is the oil flowing predominantly through the central section of the test specimen, but it flows in a finger like pattern, only occupying certain pore areas.

There are two effects superimposed here. First, there is the inlet end effect whereby one dimensional flow is not obtained. Second, there is immiscible fingering. Immiscible fingering is the process whereby the advancing front of the displacing fluid is formed of a series of separate fingers as shown in Fig. 7.15 (Kueper and Frind 1988). In homogeneous porous media, fingers develop due to microscopic heterogeneities. If stabilizing forces dominate over destabilizing forces, the fingers are damped out. However, above a certain flow rate, destabilizing forces dominate and fingers grow (Kueper and Frind 1988).

7.6 RELATIVE PERMEABILITY TEST ON SINGLE SPECIMEN

A drainage relative permeability test was conducted on another specimen with 20% clay content. This specimen had an intrinsic permeability of $2.2 \times 10^{-15} \text{ m}^2$. The results of this test are shown in Fig. 7.16. The oil relative permeabilities show an increase with increasing oil flux (decreasing number of water syringes). Again, the oil relative permeabilities are all quite high, possibly indicative of chemical reaction. To show the

Fig. 7.14 Cross section of soil core after drying in oven showing observed pattern of oil blotches.

Fig. 7.15 Immiscible fingering in porous media (after Perkins and Johnston 1969).

Fig. 7.16 Relative permeabilities as functions of the number of water syringes.

Fig. 7.17 Relative permeabilities as functions of the number of water syringes, with relative permeabilities shown on different axes.

water relative permeabilities better, the oil and water relative permeabilities are shown on separate axes in Fig. 7.17. The water values are unusual. At high water fluxes, they increase with decreasing number of water syringes, then decrease as would normally be expected.

The relative permeabilities were then combined with the saturations from the appropriate flux ratios, as shown in Fig. 7.18. The values are shown on separate axes in Fig. 7.19 to make the water values more visible. Unfortunately, the combination of the previous saturations and these relative permeabilities still did not provide smooth curves.

The problem lies with both the saturations and the relative permeabilities. The relative permeabilities in Fig. 7.17 do not form smooth curves when expressed as functions of the number of water syringes. The same values cannot be expected to form smooth curves when expressed as functions of water saturation. There is also a problem with the saturation values. Theoretically, high water to oil flux ratios should correspond to high water saturations. The saturations from the previous test are shown as functions of the number of water syringes in Fig. 7.20. There is a general trend of increasing water saturation with increasing water flux, but individual points do deviate from the trend. As a result, the saturations do not combine with the relative permeability values in a convenient manner.

Fig. 7.18 Relative permeabilities combined with previous water saturations.

Fig. 7.19 Relative permeabilities combined with previous saturations, with relative permeabilities shown on different axes.

Fig. 7.20 Water saturation as a function of the number of water syringes.

One hypothesis for the variability in the results is as follows. A Proctor density test was undertaken to determine where the target compaction moisture content was in relation to the optimum. The value of 10.3% had been selected as a value that was neither too wet nor too dry to compact properly. The results of the Proctor test are shown in Fig. 7.21. The chosen moisture content turned out to be approximately the optimum value. This is unfortunate. Lambe (1958b) showed that the structure of a clayey soil is a function of the compaction moisture content. Close to the optimum moisture content, the soil structure is very sensitive to the compaction moisture content, as shown in Fig. 7.22. Since the compaction moisture content has been close to the optimum for all of these tests, the soil structure has likely been quite variable, resulting in a variable void ratio and permeability.

In an attempt to develop slightly better results, the relative permeability test was repeated. It was hoped that better relative permeability data would overcome the individual anomalous saturation values. This test was unsuccessful, but brought to view a previously neglected issue.

After every test which has been described, the oil and water were collected from the outlet reservoir for disposal. The oil and water were put in tall glass graduated cylinders. Both the top oil phase and the bottom water phase were slightly cloudy. After a time, the top oil phase (which was exposed to the atmosphere) cleared. As a test, a sample of cloudy oil was placed in an oven at 110 °C. The oil cleared within a couple of minutes. The oil and water are obviously not completely immiscible. Some water must partition

Fig. 7.21 Results of Proctor test.

Fig. 7.22 Particle orientation and dry unit weight as functions of the compaction moisture content (after Lambe 1958b).

into the oil giving the cloudy appearance. The cloudiness of the water was attributed to the same phenomenon.

In this next test, the water was much more cloudy than in previous tests. Substantial amounts of sediment were found in the bottom of the outlet reservoir. The pressures had also considerably decreased over the time of the test. The conclusion was that piping had occurred to such an extent as to seriously alter the relative permeability.

Although the physical signs of piping were much more evident in this test than any previous test, it was possible that piping had occurred in the other tests. To evaluate this hypothesis, a lower gradient test was undertaken.

7.7 LOW GRADIENT DRAINAGE RELATIVE PERMEABILITY TEST

Another specimen with a clay content of 20% was compacted with a moisture content of 10.3% and consolidated for 24 hours. The intrinsic permeability was calculated from the water saturation data to be $2.63 \times 10^{-15} \text{ m}^2$. During saturation, the hydraulic gradient was 118. Relative permeability tests were carried out with flux ratios ($q_w:q_o$) of 6:2, 4:4, 2:6, and 0:8. During these tests, the flow rate was adjusted so that the hydraulic gradient was approximately the same as for the saturation stage. The outflow was completely clear. Piping did not occur. Due to the high oil viscosity, the flow rates had to be very small. Three weeks were required to obtain these four data points. The relative permeabilities are shown in Fig. 7.23. The oil relative permeabilities are all quite low compared to the

Fig. 7.23 Relative permeabilities as functions of the flux ratio for the low gradient test.

previous test results. The oil relative permeability curve actually slopes in the incorrect direction. The water relative permeability values are quite reasonable.

The oil relative permeabilities are all close to unity in comparison to the values from previous tests. It must be concluded that the high oil relative permeabilities were not due to chemical reaction. The apparent high oil relative permeabilities were probably products of several effects.

As described previously, the oil appeared to be flowing predominantly through the central section of the specimen. The inlet length was longer than the specimen. As well, observed blotching was likely indicative of immiscible fingering. Both the long inlet length and immiscible fingering are problems which occur when a high flow rate is used. Recall from chapter four that the inlet length increases with increasing flow rate. Recall also, from the discussion on immiscible fingering, that fingering only occurs above a critical destabilizing flow rate. The two effects result in an uneven distribution of the fluids across the cross section. The pressure differential is measured at the circumference where higher water saturations were observed. The erroneous oil relative permeabilities may result from applying this pressure drop to both fluids. The test cell design is based on one dimensional flow, which is not occurring.

In the low gradient test, the inlet length effect and immiscible fingering are probably reduced. All of the relative permeabilities are close to unity in comparison to previous

results. The true relative permeability values are probably less than unity.

An additional factor is that piping may have occurred during the tests in which high oil relative permeabilities were observed. Recall, however, that the water relative permeability at residual oil saturation was found to be 0.9. If piping had occurred, this value probably would have exceeded unity. It is more likely that the inlet end effect and immiscible fingering explain the high oil relative permeabilities.

CHAPTER 8

SUMMARY AND CONCLUSIONS

Currently, engineers studying the effects of organic chemicals on clay landfill liners have considered the problem from the perspective of chemical interaction. Hydrogeologists studying the movement of DNAPLs and LNAPLs in the subsurface have considered the problem from the perspective of multiphase flow. The objective of this research has been to combine both perspectives. The migration of nonaqueous organic chemicals through landfill liners is a multiphase flow problem. Similarly, a DNAPL or LNAPL released into the subsurface could encounter a soil with some clay content, and chemical reaction could occur. To study this problem, laboratory testing was undertaken to determine the relative permeability of a clayey soil permeated with a nonaqueous organic chemical.

A review has been provided of published literature on the topics of:

- clay-organic chemical interaction
- multiphase flow
- relative permeability testing.

A test apparatus was designed to measure the relative permeability for two phase flow through a low permeability soil. The design of this apparatus involved combining current practice in the field of clay-organic liquid interaction testing with current practice on relative permeability testing.

The relative permeability permeameter was used to measure the relative permeability for the flow of water and vegetable oil through soil with moderate to low permeability. The soil specimens were mixtures of kaolin clay and No. 70 silica sand. Initial results seemed to indicate that increases in intrinsic permeability occurred when the soil was permeated with the oil. This was recognized by oil relative permeabilities much greater than unity. However, later tests conducted at lower gradients dispute this initial conclusion. The apparent high oil relative permeabilities are likely due to nonuniform fluid distribution across the cross section. The inlet effect and immiscible fingering seem to have resulted in higher oil saturations in the centre of the specimens, and oil flowing in discrete fingers. The pressure differential used to calculate the relative permeabilities was measured at the circumference. The water saturation was higher at the circumference. Since water has a lower viscosity than oil, the pressure differential at the circumference was probably less than the pressure differential in the high oil saturation core. The use of the measured differential pressure to calculate relative permeabilities produced erroneously large values. The results of the low gradient test indicate that the true relative permeabilities may well be less than unity.

CHAPTER 9

SUGGESTIONS FOR FUTURE RESEARCH

Vegetable oil was used in the laboratory testing due to its high viscosity, inexpensiveness, and lack of noxious qualities. Now that many of the difficulties involved in this type of testing have been recognized, and some of them overcome, it would be interesting to study a more realistic problem.

The first step involved in continuing this work would be to modify the pump and the pump filling procedure to facilitate the use of noxious volatile organic chemicals. More realistic contaminants could then be used. A contaminant like TCE would be ideal. Since this chemical is a low viscosity DNAPL, it would be more likely to permeate through a water saturated landfill liner than higher viscosity chemicals, or chemicals less dense than water. From a testing perspective, TCE would be unlikely to form bubbles, as gasoline is thought to have done, since TCE is denser than water. TCE would also be more likely to fill the dispersing grooves and generate one dimensional flow. A lower permeability soil could be used due to the lower viscosity of TCE. A lower permeability soil would be more likely to react chemically with the TCE due to the higher clay content of the soil. Lower flow rates should be used to minimize the inlet end effect and eliminate immiscible fingering. These tests would need to be of long duration.

Another modification which could be made is to develop a procedure to measure the fluid

saturations during a test. A method based on the electrical resistivity of the soil specimen might be the best option. A calibration curve would have to be developed, possibly using distillation of water and TCE from the soil. This modification would allow the relative permeabilities at all flux ratios to be determined on the same soil specimen while also measuring the fluid saturations.

The effect of an unsaturated initial soil specimen might also be of interest. Landfill liners are generally above the water table. Capillary effects keep the clay near saturation. The effect of partial saturation on the relative permeability could be studied.

In conclusion, this work has attempted to bring together the two perspectives of chemical interaction and multiphase flow to study the problem of the movement of organic chemicals through soil more completely. There is, however, much work that remains to be done.

CHAPTER 10

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APPENDIX A: SHOP DRAWINGS

CARLETON UNIVERSITY
DEPT. OF CIVIL AND
ENVIRONMENTAL ENG.

PROJECT	CLAY RELATIVE PERMEABILITY
DATE	TOP PLATE
DESIGN	STAINLESS STEEL
REV.	1
DESIGNED BY	M. CUNNINGHAM
DATE	AUG. 11, 1993
CHECKED BY	SEPT. 21, 1993
NOTES	1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SPECIFIED

P2

CARLETON UNIVERSITY
DEPT. OF CIVIL AND
ENVIRONMENTAL ENG.

CLAY RELATIVE PERMEABILITY

Piston Head

STAINLESS STEEL

1

M. CUNNINGHAM

AUG. 11, 1993

DEC. 17, 1993

1. ALL DIMENSIONS ARE IN INCHES UNLESS OTHERWISE SPECIFIED

P3

**CARLETON UNIVERSITY
DEPT. OF CIVIL AND
ENVIRONMENTAL ENG.**

DATE RELATIVE TO REFERENCE
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CARLETON UNIVERSITY
DEPT. OF CIVIL AND
ENVIRONMENTAL ENG.

CLAY RELATIVE PERMEABILITY

CYLINDER SECTION VIEWS

MC 901 NYLON

1

M. CUNNINGHAM

SEPT. 15, 1993

NOV. 17, 1993

1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SPECIFIED

P5-S

SEC. A-A

CARLETON UNIVERSITY DEPT. OF CIVIL AND ENVIRONMENTAL ENG.	
CLAY RELATIVE PERMEABILITY	
SYNTHETIC BASE PLATE	
STAINLESS STEEL	
1	
IN. CLAY	
ALC. 25. 1993	
JAN. 4. 1994	
1. 10. 1994	
P6

O-RING GROOVE DETAIL

CARLETON UNIVERSITY
DEPT. OF CIVIL AND
ENVIRONMENTAL ENG.

PROJECT CLAY RELATIVE PERMEABILITY

SYRINGE CYLINDER

MC 901 NYLON

8

M. CUNNINGHAM

SEPT. 7, 1993

FEB. 8, 1994

1. THE DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SPECIFIED

P7

CARLETON UNIVERSITY
DEPT. OF CIVIL AND
ENVIRONMENTAL ENG.

CLAY RELATIVE PERMEABILITY

SYRINGE PLUNGER

1 - 1/2 INCH DIAMETER
BRASS ROD

8

M. CUNNINGHAM

SEPT. 7, 1993

FEB. 8, 1994

1. NO LIABILITY FOR INACCURACIES OR OMISSIONS

P8

CARLETON UNIVERSITY
DEPT. OF CIVIL AND
ENVIRONMENTAL ENG.

CLAY RELATIVE PERMEABILITY

SYNTHETIC CLAMPING PLATE

ALUMINUM

1

M. CUNNINGHAM

SEPT. 7, 1993

JAN. 4, 1994

1. AS SHOWN ON A SEPARATE SHEET

P9

CARLETON UNIVERSITY DEPT. OF CIVIL AND ENVIRONMENTAL ENG.	
PROJECT	CLAY RELATIVE PERMEABILITY
DATE	STUB COLUMN SPACER
DESIGN	STEEL
BY	1
DESIGNED BY	M. CUNNINGHAM
DATE	SEPT. 9, 1993
DATE	SEPT. 21, 1993
DATE	1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SPECIFIED
DATE	P11

1/8 INCH POROUS PLASTIC

CARLETON UNIVERSITY
DEPT. OF CIVIL AND
ENVIRONMENTAL ENG.

CLAY RELATIVE PERMEABILITY

PIEZOMETER RING

70 MICRON POROUS PLASTIC

2

M. CUNNINGHAM

OCT. 21, 1993

OCT. 21, 1993

P12

END

0 | 2 + 0 6 + 9 | 5

FIN